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A SERIES OF TEXTBOOKS FOR PERSONS ENGAGED IN ENGINEERING PROFESSIONS, TRADES, AND VOCATIONAL OCCUPATIONS OR FOR THOSE WHO DESIRE INFORMATION CONCERNING THEM. FULLY ILLUSTRATED

FOUNDATIONS
RETAINING WALLS
CULVERTS
TUNNELS
DAMS
INTRODUCTION TO CONSTRUCTION DRAWING

SCRANTON:
INTERNATIONAL TEXTBOOK COMPANY
1924

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Press of
International Textbook Company
Scranton, Pa.

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FOUNDATIONS

(PART 1)

INTRODUCTION

1. Definitions.—It is obvious that the weight of any artificial structure must be, directly or indirectly, transferred to and carried by some part of the natural surface of the earth. The word *foundation*, as used in connection with engineering structures, is commonly applied both to the natural material, or the portion of the earth's surface, on which a structure rests, and to the lower part of the structure itself that is in contact with or contiguous to the natural surface. To prevent the confusion likely to result from this indefinite use of the word, various distinguishing terms have been employed. Some writers call the natural surface on which the structure rests the **foundation bed.** Here, however, the term *subfoundation*, which seems appropriate, will be used instead of *foundation bed*, and the following definitions will be adopted:

The **subfoundation** of a structure is that part of the natural surface of the earth on which the structure rests.

The **foundation** of a structure is not only that part of the structure that is directly in contact with the subfoundation, but also as much of the lower part of the structure as may have to be modified to properly connect the structure with the subfoundation. Thus, if a bridge pier rests on level solid rock, equal in strength to the material of which the pier itself is composed, it may be safely designed as shown in Fig. 1, in which case the foundation is restricted to, at most, the

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first course of masonry. But if it is necessary to distribute the weight of the structure over a larger area of subfoundation, as much of the lower part of the structure as must be modified to comply with this requirement may be properly called a part of the foundation. Thus, in Fig. 2, all that part of the structure from *a* up to *b* may be properly called the foundation.

The definition, sometimes given, that the foundation is that part of a structure that lies below the natural surface of the ground, or below the surface of the water, is indefinite and inaccurate.

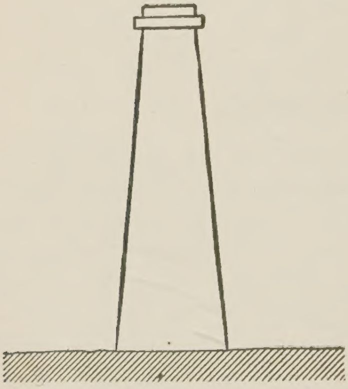
FIG. 1

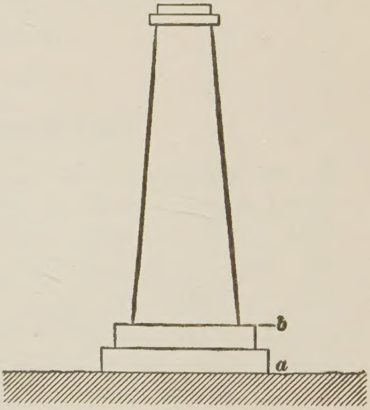

FIG. 2

2. Main Factors Governing the Design of Foundations.—The stability and life of an engineering structure depend largely on the character of its foundation. The selection and designing of suitable foundations is, therefore, one of the most important and responsible duties of engineers. The requirement of first importance is that the foundation should be of sufficient strength to support permanently the proposed structure. On the other hand, sound economy requires that no more money should be spent on the foundation than intelligent skill and prudence may dictate. It is not sufficient that the engineer may be able to show that a foundation designed by him is unquestionably secure; he should also be able to prove that an adequate foundation could not have been provided with less expenditure of money and time.

The great variety of materials and conditions that are met with in foundation work calls for the exercise of good sense and sound judgment, as well as for the application of technical knowledge. Foundation problems do not always lend themselves to exact mathematical solution to the same degree as, for instance, the designing of a bridge truss, where the properties of the steel are comparatively uniform and well known, and the magnitude and distribution of all the stresses can be determined with precision. It is none the less true, however, that the intelligent solution of a problem relating to foundations requires a thorough knowledge of mechanical principles, and the ability to apply them properly.

THE SUBFOUNDATION

MATERIALS

CLASSIFICATION

3. The subfoundation of a structure may consist of any of the materials that are found on the earth's surface and are suitable for the purpose. The materials usually regarded as suitable are: (1) *solid rock*, including shale in its natural geological position; (2) *loose rock*, which is rock broken into masses of comparatively small sizes; (3) *earth*, including clays, loams, and bogs, in whatever condition they may be found; (4) *sand*, including gravel.

In some cases, two or more, or even all, of these materials may be met with in the same subfoundation.

ROCK SUBFOUNDATION

4. Rock in its undisturbed geological position forms the best subfoundation, and is always to be preferred when it is available. For important structures of great weight, rock subfoundations are considered almost indispensable. Nearly any sound durable rock, including shale of average hardness, found in its natural position, may be safely used for the support of any ordinary structure.

5. Crushing Strength of Solid Rock.—The supporting power of a rock subfoundation may be considered as approximately equal to the resistance to crushing of the material of which the rock is composed, modified by a suitable factor of safety. Samples selected for crushing tests are likely to be superior to the average, and as, in large areas, imperfections are likely to exist, the factor of safety should be a liberal one: it should not ordinarily be less than 8, nor, except in rare cases, more than 15. If the rock appears to be of uniform good quality, without cracks or seams, over and for a considerable space around the area to be occupied by the foundation, a load equal to one-tenth of the ultimate crushing strength of the material may be confidently used, subject to the precautions hereafter mentioned. Crushing tests are usually made on small cubes, and the crushing strength is stated in pounds per square inch of the surface exposed to pressure. The units commonly applied to foundations are the square foot and the ton (2,000 pounds), and it is usually assumed that the strength per square foot of any material is 144 times the crushing strength per square inch.

TABLE I

CRUSHING STRENGTH OF ROCK

Kind of Rock	Ultimate Crushing Strength					Safe Foundation Load, Tons per Square Foot Factor of Safety of 10		
	Pounds per Square Inch		Tons per Square Foot					
	From	To	From	To	Average	From	To	Average
Granite . .	10,000	20,000	720	1,440	1,080	72	144	108
Limestone .	6,000	18,000	430	1,300	870	43	130	87
Sandstone .	4,000	15,000	300	1,080	690	30	108	69
Shale . . .	400	14,000	30	1,010	520	3	100	52

The crushing strength of the different kinds of rock, as well as of different varieties of the same rock, varies within wide limits. Table I gives the approximate crushing strength of the various kinds of rock more frequently met with, and the foundation loads that they may usually be depended on to carry safely under normal conditions.

LOOSE-ROCK SUBFOUNDATIONS

6. Description of Loose Rock.—Under the operation of natural laws, the solid strata or deposits of rock have, in many cases, been broken up, and the resulting débris moved from their original position by the action of gravity, water, and ice. The individual masses of this broken-up material may be of any size or form. Masses of considerable size and angular shape are commonly called **loose rock,** while smaller fragments that have become rounded by attrition are called **boulders.** In a deposit of such materials, there will exist cavities between the separate masses of rock, which may be voids or may be filled with smaller fragments of the same material, or with earth or sand. If filled with a material that has hardened into rock and has cemented the fragments into a solid mass, the rock is called **breccia, conglomerate,** or **pudding stone.**

7. Loose Rock as a Subfoundation.—Loose rock in any of its forms may make a satisfactory subfoundation. The celebrated cantilever bridge at Niagara Falls rests on a loose-rock subfoundation composed of large masses of rock detached from the adjoining cliffs, the spaces between the large fragments of rock being filled with smaller fragments and with earth. The pressure on this subfoundation is about 3 tons per square foot. Subfoundations of this kind, owing to the comparatively recent geological age of the formations and the conditions under which the deposits have been made, require careful examination, and, if possible, should be avoided for important structures.

EARTH SUBFOUNDATIONS

8. General Description.—Under the general head of earth are here included the different clays, loams, and other soils, in whatever condition they may be found. In hardness and capacity to sustain weight, they vary all the way from the indurated clays and hard pans to soft mud, often mixed with more or less of the organic matter found in swamps and bogs. Nearly all these materials are porous in structure, and, when confined and subjected to sufficient pressure, are compressed to a greater or less degree. When moist or wet, they are generally *plastic;* that is, they possess the property of yielding or flowing when exposed to pressures exceeding certain limits, these limits being different for different materials. This property of yielding under pressure is so characteristic of the class of materials under consideration, that the term "compressible materials" is generally applied to them in connection with foundation work. The different degrees to which these materials are compressible, and the difficulties and dangers that this property introduces into the problems of earth subfoundations, make them a subject of great interest and importance to the engineer.

It sometimes becomes necessary, when solid rock is inaccessible, to found important structures on clay or earth of a quality far from satisfactory to the engineer, and he will often find his knowledge, experience, ingenuity, and skill taxed to the utmost by the problems presented. For the less important, and in fact for the great majority of ordinary structures, earth subfoundations, if intelligently made use of, may be safely relied on.

9. Strength.—When exposed to increasing pressure, as in testing machines, clay and earth usually yield gradually by flow and deformation of the mass, rather than by distinct crushing, and for this reason crushing tests on small samples are of comparatively little value. Their strength is largely affected by the quantity of water they contain; and the extent to which they may be exposed to water in the subfoundations is an important element to be considered in determining their sustaining capacity.

The data as to the weight that may safely be placed on earth subfoundations are derived largely from observation and experience. Indurated clay, called **hard pan**, and ordinary clays in comparatively dry situations may usually be depended on to carry a weight of from 1.5 to 2 tons per square foot of surface. In some standing structures founded on clay, the weight on the subfoundation is greatly in excess of these figures, and no settlement or indications of failure have been observed. However, excessive loads on clay subfoundations, even where tests indicate that the structures may be safe, are not sanctioned by good practice.

TABLE II

SAFE LOADS ON EARTH SUBFOUNDATIONS

Kinds of Material	Loads in Tons per Square Foot	
	From	To
Hard pan and other indurated clays	2	$2\frac{1}{2}$
Ordinary clays and clay soils, not submerged in water	$1\frac{1}{2}$	2
Clay, soft and plastic	$\frac{1}{2}$	1
Ordinary soils, comparatively dry	1	$1\frac{1}{2}$
Ordinary soils, wet	$\frac{1}{4}$	1
Swamp and bog material	$\frac{1}{8}$	$\frac{1}{2}$

Experience seems to indicate that the strength of earth subfoundations of the various general kinds may be taken approximately as given in Table II. Owing, however, to the greatly varying individual character of materials that may be classed under the same general name, the engineer must in each case be guided largely by judgment based on experience and actual tests.

SAND SUBFOUNDATIONS

10. Ordinary Sand and Gravel.—Sand and gravel are but slightly compressible, even when saturated with water, and are capable of carrying very great loads. The high frictional resistance of the separate grains or pebbles on each other tends to prevent the movement or flow that is characteristic of clay. The material is, however, readily eroded by flowing water, and, when utilized for subfoundations, great care must be taken to protect it from direct contact with currents of water, whether of ordinary streams or those caused by wave action. Sand and gravel are both likely to undergo slight initial compression when subjected to pressure; this fact should be taken into consideration, and proper allowance should be made for it in designing structures to rest on this kind of subfoundation. Clean dry sand can bear a load of from 2 to 4 tons per square foot.

11. Quicksand.—One of the most treacherous and troublesome materials with which the engineer has to deal in foundation work is **quicksand.** It is difficult to give a satisfactory definition of this material, though its physical properties are easily described. In most respects it does not differ essentially from ordinary fine sand, and when removed from its native bed and freed from water it cannot usually be distinguished from sands that do not possess its peculiar properties. The separate grains are always small and generally more rounded than those of ordinary sand. But when saturated with water, a mass of this material seems to lose more or less of its internal cohesion or friction, and its power to support weights is almost zero. A bar may be worked to the bottom of a bed of quicksand several feet deep with very slight exertion, and men and animals attempting to walk over such a deposit are often hopelessly engulfed. This description applies to well-developed examples of the material. There is no well-defined demarcation between ordinary sand and quicksand, and the one may pass into the other by imperceptible stages; in other words, sands may possess the peculiar properties of quicksand in very different degrees.

In general, quicksand has practically little or no value as a subfoundation; however, it may sometimes be so treated as to make it serve to support ordinary structures. If a bed of quicksand is drained, it loses its distinctive property; it becomes more compact and resisting than ordinary sand, and may make a convenient and secure subfoundation.

SUBFOUNDATIONS OF MIXED MATERIALS

12. It is often found that the site of a structure is occupied by several of the materials described—either in separate deposits or mixed together in various proportions. If rock is found, it may extend over only a part of the area of the foundation, or it may contain large fissures filled with other materials; or the rock, although extending over the whole area, may not be all of the same kind or quality, in which case different parts of the subfoundation require different methods of treatment. Again, the area may be occupied by a mixture of earth and loose rock or boulders, or part of it may be clay and part of it sand or quicksand. Such complications are of frequent occurrence, and conditions are always likely to be found that are without precedent in the engineer's experience, and that will tax to the utmost his judgment and skill.

EXAMINATION AND TESTS OF SUBFOUNDATIONS

13. It is generally unsafe to trust to surface appearances in judging of the character, strength, and soundness of subfoundations. A stratum of rock may, when its surface is uncovered, appear to be of satisfactory character, without serious fissures or other defects, and may give the impression of being continuous and solid downwards to an indefinite depth. This appearance may, however, be deceptive; the layer of rock may be comparatively thin, and may be underlaid with a body of soft clay or other unresisting material, and may therefore not be capable of sustaining the structure designed to rest on it. Not a few important engineering structures have failed from the existence of such unforeseen conditions. Likewise, an apparently satisfactory bed of clay may be too thin, or may be underlaid with a stratum of sand that the swift current of a stream may sometime reach and carry away, undermining the clay and causing the structure founded on it to fail. It is therefore necessary that the engineer should carefully investigate not only the material that he purposes to use for a subfoundation, but, as far as may be possible, all the conditions surrounding it, and satisfy himself that these conditions are not such as may ultimately result in the destruction or impairment of the proposed structure.

EXAMINATION AND TESTS OF ROCK SUBFOUNDATIONS

14. Geological Examination.—A general knowledge of the geology of the region—such as the character and succession of the local formations, the inclination or dip of the strata, and the prevalence of faults and other disturbances—should be acquired, and this knowledge should be utilized in the study of the immediate locality of the work. In a region where the geological formations have been subjected to upheaval and are greatly broken up or much inclined, greater caution is necessary than where the strata remain in their original nearly horizontal and unbroken condition.

Where the structures to be erected are of secondary importance and the weight to be carried is comparatively small, the character of the subfoundation may often be determined accurately enough from the nature of the neighboring strata and their outcrop in near-by streams or ravines. For all important works, however, special tests should be made.

15. Drill Test and Drilling Machines.—Where the subfoundation is rock, it may be most satisfactorily examined by sinking drill holes into it. The number and depth of these drill holes depend on the apparent conditions and the importance of the structure. In the case, for instance, of a bridge with short spans carried on comparatively low piers, the same thoroughness of investigation is not called for as in the case of a long suspension bridge with high piers, carrying very heavy loads. In the former, one or two drill holes on the site of the pier, carried to a depth of 10 feet, or even less, might be sufficient; while in the latter, drill holes not only within, but for some distance outside of, the boundaries of the pier and carried to a depth of from 20 to 30 feet, or more, may be advisable.

16. In the great majority of cases, drill holes of the size of, and made with the apparatus commonly used for, those in quarry and rock-excavation work, will answer every purpose. In the more important works, particularly where there is reason to expect defects and irregularities, and where the holes are to be carried to a considerable depth, some one of the forms of drilling apparatus that yield complete samples, or **cores**, of the material passed through should be used. These machines are generally power driven. The drilling tool is a hollow rod that, when revolved, makes an annular cut into the rock, leaving an undisturbed section, or core, within the tool; this core is occasionally broken off and brought up with the tool. The recovered cores may, after examination and record, be preserved for future reference.

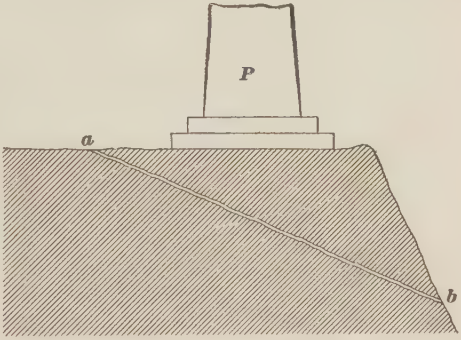

FIG. 3

17. Test drilling will, if properly conducted, disclose any subterranean defects in the subfoundation, and will furnish data for a second judgment of the capacity of the material. The defects that may thus be disclosed are: (1) beds or pockets of clay or other soft material dangerously near the surface; (2) large cavities that might weaken the subfoundation; and (3) thin layers of clay or argillaceous shale, which, in cases where the strata are much inclined and other conditions permit. might allow the structure and its foundation to slide or move laterally. Such a possible condition is illustrated in Fig. 3. The weight of the pier *P* and its load might cause the subfoundation to slide along the inclined joint *a b*, particularly if this joint is lubricated by a thin layer of unctuous clay. This is not a purely supposititious case, since the destruction of not a few structures has been caused by similar conditions.

EXAMINATION AND TESTS OF OTHER SUBFOUNDATIONS

18. All that has been said of rock subfoundations applies with equal force to loose rock, earth, and sand subfoundations; in fact, these usually require the exercise of greater care and caution than ordinary rock foundations. The formations consisting of these materials, as they are now found, are much more recent geologically than rock formations, and are far less stable in character. It is not at all uncommon to find beds of firm strong clay that prove comparatively thin and are underlaid with deposits of much softer clay, alluvium, or quicksand, saturated with water and having very little capacity to carry loads. Two methods are in common use for testing such subfoundations; namely, *test pits* and *borings*.

19. Test pits, or **wells,** are ordinary excavations similar to the common well, carried to such depth as may be thought necessary, within or near the boundary of the foundation. The method of sinking such test pits requires no special description.

20. Borings.—The method of boring is, in general, similar to the rock drilling described in Art. **16,** but is usually less expensive and more expeditious. The bore holes are really wells or test pits of a small diameter. Several methods of making such borings are in use, some of which are so simple as to require no description here. Frequently, the procedure and the apparatus used are almost the same as for rock drilling. This method is objectionable because of the difficulty of obtaining good samples and determining the character of the materials passed through, but it may give results that are satisfactory in many cases.

Thus, a drill hole sunk into a bed of clay or other material may show that no important change in its character occurs within such a distance from the surface as would render it unsuitable for foundation purposes. But even if the results from this method of making borings were always satisfactory, difficulties are frequently met with that make it inapplicable. A layer of sand or other soft material may be encountered that has not sufficient stability to maintain the wall of the bore hole; the material caves or runs into the hole, filling it and obstructing the operation of the drill. To prevent this, the hole must be *cased;* this is done by driving down, inside the hole, an iron tube. When borings are to be carried to a considerable depth, it is generally best to begin with the expectation that casing will be necessary, and to make the bore hole large enough to allow the use of iron pipe not less than 3 inches inside diameter.

21. For these larger holes, two well-defined methods of boring are employed. In the one, suitable earth augers are used, which excavate and bring up the material in its natural condition; in the other, called **wash boring,** the material is loosened partly by the auger or drill, and partly by a jet or current of water, which also carries the loosened material to the surface. In this method, the drill rod is tubular, and a current of water is forced through it by pumps or other devices. The water escapes at the working end of the drill and rises through the annular space between the drill and the casing, carrying with it the excavated material, which may be collected and examined. As the drill progresses downwards, the casing is, from time to time, forced downwards by a pile driver or other suitable device, additional lengths of pipe being added by means of screw joints or couplings, as required. The first method yields the most satisfactory samples of the material passed through; the second is more expeditious, and, with a proper equipment, more economical. Whatever method may be employed, the results of borings should be carefully observed, accurately recorded, and samples preserved for future reference.

DESIGN OF THE SUBFOUNDATION: THEORY OF PRESSURE

22. Required Area of Subfoundation.—In the case of foundations for ordinary structures, where weight is the only force to be resisted, and where that weight is uniformly distributed over the whole of the subfoundation, the first problem of importance will be: Over what area of the subfoundation must the weight be distributed, in order that the safe bearing capacity of the material of the subfoundation shall not be exceeded?

Where the material is solid rock in its natural geological position, it is usually unnecessary to solve this problem, since almost any sound rock is sufficiently strong to carry the concentrated weight of any well-designed structure. But where earth or sand subfoundations must be utilized, care should be taken to distribute the weight to be carried over such an area that the resistance of the material shall safely exceed the applied weight. Thus, if the estimated weight of a bridge pier is 1,200 tons, the weight that may come on it from the dead load of the bridge, 500 tons, and the weight of expected live load, 300 tons, the subfoundation will be depended on to sustain a load of 2,000 tons. If the subfoundation is clay and may be safely loaded with 2 tons per square foot, the area required will evidently be 2,000 ÷ 2 = 1,000 square feet.

Example.—The foundation for a stand pipe has the form of a frustum of a cone, the dimensions being as follows: diameter of upper base, 24 feet; diameter of lower base, 30 feet; height, 5 feet. The weight of the material of the foundation is 100 pounds per cubic foot, and the weight of the stand pipe is 500 tons. If the subfoundation is clay that may be loaded with 2 tons per square foot, what is the required area?

Solution.—The weight of the foundation is (see *Geometry*, Part 2)

$$\left(24^2 \times .7854 + 30^2 \times .7854 + \sqrt{24^2 \times .7854 \times 30^2 \times .7854}\right) \times \frac{5}{3} \times 100$$
$$= 287{,}460 \text{ lb.} = 143.73 \text{ T.}$$

The total load on the subfoundation is 500 + 143.73 = **643.73 T.** The required area of the subfoundation is, therefore,

643.73 ÷ 2 = 321.87 sq. ft., nearly. Ans.

23. If the plan of the structure is irregular, so that it is difficult to secure a uniform intensity of pressure over the whole subfoundation area, or if lateral forces must be taken into consideration, or if the character or sustaining capacity of the material of the subfoundation varies over parts of it, the question becomes more complicated. Clay and earth subfoundations are usually more or less compressible; the amount of compression or settling is in direct proportion to the intensity of the applied pressure, and a slight variation in the settlement of different parts of the subfoundation may distort or disrupt the structure erected on it.

A not uncommon example of a load not uniformly distributed is a masonry arch culvert built on a compressible subfoundation under a high railroad embankment, as illustrated in Fig. 4. The weight on the subfoundation varies with the height of the embankment over it, being greatest directly under the middle. If the foundation of such a culvert is made of the same width throughout, the intensity of pressure on it will increase as the middle section is approached, unequal settlement may follow, the structure may take the distorted form shown by the dotted lines, and the distortion may be sufficient to fracture the masonry. If the area of the subfoundation is increased properly from the ends toward the middle of the structure, so as to distribute the load evenly per unit area, then, whatever settlement may take place will be the same throughout the whole length of the structure, and the masonry will not be injured. The walls of heavy buildings are often cracked through the failure of the designer to properly proportion the area of the subfoundation to the pressure on it

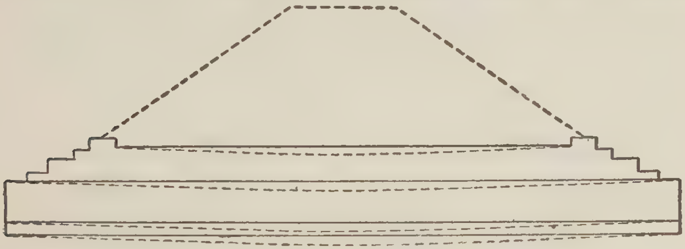

Fig. 4

It is always desirable to avoid subfoundations of compressible material, or, if this cannot be done, to make the loading so light that settlement, if any, will be so slight as to be negligible. But as this is not always possible, and as it can seldom be determined in advance whether a material will be compressed under a given load, and, if so, how much, it is important that in earth subfoundations every possible effort should be made to distribute the pressure as evenly as possible over the whole area covered.

24. Depth of Subfoundation Below Surface of Ground.—Foundations in earth should be carried to such a depth below the ground surface that frost will not reach them. Nearly all moist earth expands, or heaves, with freezing, and repeated freezing is likely to soften and disintegrate it. It may also be subject to other disturbances near the surface. The depth of foundations may be dictated by conditions other than frost. Often a good material cannot be found except at greater depths than are necessary to provide against frost.

The penetration of frost varies with the latitude. In the American Gulf States, ice seldom forms; while in the Lake region, the ground sometimes freezes to a depth of 5 or even 6 feet. Ordinarily, in the northern parts of the United States, subfoundations 4 feet below the ground surface may be considered safe from injury by frost.

25. Effect of Weight and Friction of Superimposed Earth.—Not only is the soil likely to be firmer and harder a few feet below the surface, but the surrounding earth, by its weight, tends to counteract any movement and therefore to increase the bearing capacity of the subfoundation. Thus, if a wall is founded at a depth of 5 feet below the surface, the weight of the surrounding earth above the subfoundation tends to prevent any flow of the clay from under the foundation. Such movement of the material can only take place by overcoming the weight and internal friction of the banks of clay around the foundation, resulting in the upheaval of the surface around the structure.

This counter pressure of the superimposed earth is a factor of much importance in the strength of subfoundations in plastic soil, but its exact value is difficult to compute. If the soil were a fluid, like water, the upward pressure under the foundation would be equal to the weight of a column of fluid having a height equal to the depth of the subfoundation below the surface and a cross-section equal to the area of the foundation. But, in clay, this pressure is modified by the internal friction of the material. The problem of pressures in plastic or semifluid materials like clay has been studied by physicists, and formulas to express its value have been devised, but the results do not seem to agree closely with observed facts, and these formulas must therefore be used with caution.

Rankine's formula is perhaps most often referred to by engineers. It is as follows:

$$p = w\,h\left(\frac{1 + \sin Z}{1 - \sin Z}\right)^2$$

in which p = intensity of pressure that the material can bear;
w = weight of material per unit of volume;
h = depth of subfoundation below surface;
Z = angle of repose of the material.

If h is in feet and w in pounds per cubic foot, p will be in pounds per square foot.

It should be borne in mind that Rankine's formula, although the best that has been proposed for this purpose, is but a rough approximation, and should be used only as a guide. In all important work, the bearing capacity of the soil should be ascertained by direct experiment. Thus, to determine the safe bearing capacity of the clay in which the walls of the Capitol at Albany, New York, are founded, rectangular pits about 3 feet square and 3 feet deep were excavated, and in them were built columns of masonry 3 feet square, on which various trial loads were placed. Under a load of 2 tons to the square foot, the clay subfoundation was slightly compressed, but no indications of flow were observed. When the load was increased to 6 tons per square foot, the clay under the pillar yielded very perceptibly and began tc heave up around the pillar, as shown by the dotted line in Fig. 5.

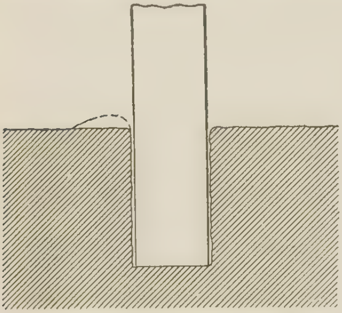

FIG. 5

The facts just explained illustrate a point that seems to be often overlooked in designing heavily loaded foundations in compressible soils. Referring to Fig. 6, which represents a structure founded on clay on a sloping hillside, it is evident that the high subfoundation wall of clay on the upper side of the structure will exert a greater pressure to counteract the flow of the clay from under the foundation than will the low wall on the lower side; that, under loads that might cause yielding on the lower side, the foundation might settle there, while remaining firm on the upper side. In such situations, care should be taken to keep the loads on the whole of the foundation within such safe limits that the clay where least confined or counterbalanced will not yield by flowing. If, for any reason, this is not possible, and loads of questionable magnitude must be used, the danger may be averted by dressing down the high bank to a slope less than the angle of repose of the material, as illustrated by the dotted line in Fig. 6. There is reason to believe that in not a few cases where heavily loaded foundations have settled unevenly, this has been due to the unequal counterbalance afforded by the superimposed subfoundation walls.

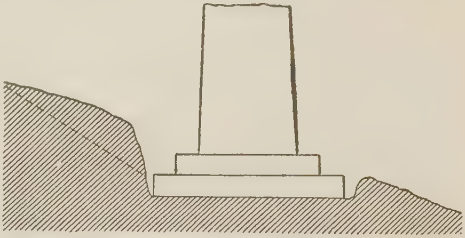

FIG. 6

EXAMPLE 1.—Taking the angle of repose of a certain semifluid soil as 31°, and the weight as 100 pounds per cubic foot, what must, according to Rankine's formula, be the depth of the subfoundation below the surface that its bearing value may be 4 tons per square foot?

Solution.—The formula of Art. **25**, solved for h, gives

$$h = \frac{p}{w}\left(\frac{1 - \sin Z}{1 + \sin Z}\right)^2$$

Here, p = 4 T. per sq. ft.; w = 100 ÷ 2,000 = .05 T. per cu. ft.; Z = 31°; sin Z = .515. Therefore,

$$h = \frac{4}{.05}\left(\frac{1 - .515}{1 + .515}\right)^2 = 8.20 \text{ ft.}\quad \text{Ans.}$$

Example 2.—The depth of a subfoundation is 12 feet below the surface. (*a*) If the angle of repose of the material is 14½°, and the weight is 110 pounds per cubic foot, what is the bearing value of the material? (*b*) If the weight of the structure, including the foundation, is 350 tons, what is the area required?

Solution.—(*a*) To apply the formula of Art. **25**, we have $w = 110$; $h = 12$; sin Z = sin 14° 30′ = .250. Substituting in the formula,

$$p = 110 \times 12 \left(\frac{1 + .250}{1 - .250}\right)^2 = 3{,}667 \text{ lb. per sq. ft.}\quad \text{Ans.}$$

(*b*) The weight of the structure is 350 T.; therefore, the area required is

$$\frac{350 \times 2{,}000}{3{,}667} = 191 \text{ sq. ft.}\quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. The angle of repose of a certain soil is 20°, and its weight is 100 pounds per cubic foot. What must be the depth of the subfoundation below the surface that its bearing value may be 3 tons per square foot? Ans. 14.42 ft.

2. The angle of repose of a certain soil being 25°, its weight, 110 pounds per cubic foot, and the depth of the subfoundation 10 feet, what will be its bearing value? Ans. 6,659 lb. per sq. ft.

3. The angle of repose is 30°; the weight of the soil, 100 pounds per cubic foot; and the depth of the subfoundation, 8 feet. (*a*) What is the bearing value of the subfoundation? (*b*) If the weight of the structure, including the foundation, is 300 tons, what is the area required?

$$\text{Ans.}\begin{cases}(a)\ 3.6 \text{ T. per sq. ft.}\\(b)\ 83 \text{ sq. ft.}\end{cases}$$

26. Effect of Lateral Forces.—Many important engineering structures—particularly dams, retaining walls, and abutments—are subjected to lateral forces in addition to weight. These may change the magnitude, and always change the distribution, of the total pressure on the subfoundation. A dam may serve as an illustration. Here, the forces that act on the structure are the weight of the structure itself and the pressure of the water against its inner face.

Let *A B E D*, Fig. 7, be a cross-section of a dam resting on the subfoundation *A B*. For convenience, a portion of the dam 1 foot long will be considered. The center of the base is at *c*, and the center of gravity of the part considered is at *g*. The pressure *P*, normal to the inner wall *B E*, exerted by the water can be determined by the methods explained in *Hydrostatics;* also, the point of application or line of action of *P*. The resultant *R* of the pressure *P* and the weight *W* of the structure gives the resultant pressure acting on the subfoundation; its line of action *a d* meets the subfoundation at *e*, which is the **center of pressure,** or point of application of the resultant pressure. If the center of pressure were at *c*, the center of the base, the weight would obviously be evenly distributed over the whole subfoundation. But with the center of pressure moved to *e*, the pressure will not be uniform; it will obviously be greater at *A* than at *B*.

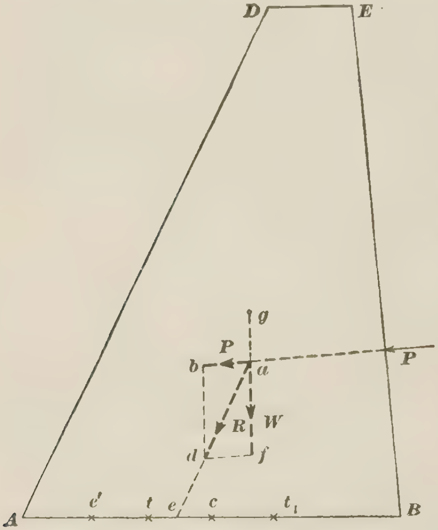

Fig. 7

27. Rule of the Middle Third.—If the base *A B*, Fig. 7, is divided into three equal parts *A t*, $t\,t_1$, $t_1 B$, the center part $t\,t_1$ is called the **middle third.** It is a principle of the statics of structures and foundations that, in order that neither end of the subfoundation or of the structure itself may be subjected to an uplifting force, the center of pressure for a rectangular subfoundation should fall within the middle third of the subfoundation. If the center of pressure is outside the middle third, as at *e'*, Fig. 7, the farther end *B* of the base will have a tendency to rise; there will, therefore, be tension instead of compression at *B*—a condition that should not obtain in masonry construction.

28. Intensity of Pressure.—In general, let *A B*, Fig. 8, be a rectangular subfoundation, or a course of masonry on which a structure or part of a structure rests. The base *A B*, whose length, in feet, is denoted by *L*, is trisected at *t* and t_1, and bisected at *C*. Let *V* (pounds) be the vertical component of the total pressure acting on *A B*; and let the distance *e C* of the point of application of this pressure from the center *C* be denoted by *d*. Then, the greatest intensity of pressure p_1, in pounds per square foot, will occur at *A*, and the least intensity p_2 will occur at *B*. It can be shown by the use of advanced mathematics that p_1 and p_2 are given by the formulas:

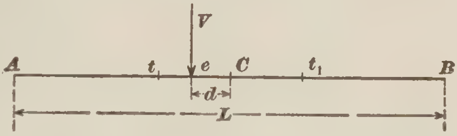

FIG. 8

$$p_1 = \frac{V}{L} + \frac{6\,V d}{L^2} \qquad (1)$$

$$p_2 = \frac{V}{L} - \frac{6\,V d}{L^2} \qquad (2)$$

The condition for which there is no pressure at *B* is obtained by making $p_2 = 0$. Putting the second member of formula 2 equal to zero, and solving for $\frac{V}{L}$, there results

$$\frac{V}{L} = \frac{6\,V d}{L^2};$$

whence

$$d = \frac{L}{6},$$

and, therefore,

$$A e = \frac{L}{2} - d = \frac{L}{2} - \frac{L}{6} = \frac{L}{3} = A t$$

That is, the point e coincides with t. If d is greater than $\frac{L}{6}$, or $A e$ less than $\frac{L}{3}$, $\frac{6\,V d}{L^2}$ is greater than $\frac{V}{L}$, and p_1 becomes negative, which indicates tension instead of pressure at B. Hence the middle-third rule given in Art. **27.**

PREPARATION OF THE SUBFOUNDATION

29. Rock Subfoundation.—The preparation of the subfoundation is generally a very simple matter, involving only manual labor. In rock subfoundations, it is not usually considered necessary to excavate below the surface to a greater depth than may be found needful to remove disintegrated and weather-worn rock, and to bring the surface into proper shape to receive the foundation. Where a stone-masonry structure is to rest directly on a sloping-rock subfoundation, the latter must be leveled off to receive the courses of masonry, or else cut into benches or terraces, according to circumstances. The dressing of the rock into benches may sometimes be avoided by the use of concrete for leveling up inequalities between the rock and the masonry, as illustrated in Fig. 9. This is, however, permissible only where the general slope of the rock is not sufficient to induce or permit sliding of the concrete on the surface of the bed rock, caused by gravity or by lateral forces acting in the direction of the downward inclination. In the case of dams, retaining walls, abutments, etc., subjected to great lateral stress and liable to failure by sliding on the subfoundation, sloping surfaces must be avoided. One of the rules commonly laid down by writers on foundations is that the surface of the subfoundation should be perpendicular to the resultant of all the forces acting on the structure. This rule

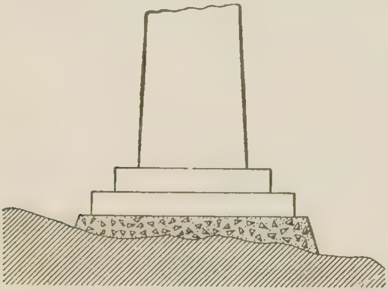

FIG. 9

is correct in principle, but it may often be deviated from within safe limits for economic reasons, as will be seen further on.

30. Earth Subfoundation.—In the preparation of earth subfoundations, the material is excavated to a sufficient depth to guard against the action of severe frost, and the whole area is graded to a level plane. In very soft and compressible material, measures are sometimes taken to solidify or reinforce the subfoundation before the structure is begun. This may be accomplished by driving a large number of short wooden piles over the area, to compress the soil into a more compact mass; but, unless these piles are below the permanent surface of the water, they may in time decay and become useless. To avoid this possibility, piles are sometimes driven and at once pulled out, and the holes are filled up with clean sand or gravel. Sand, dry earth, or crushed rock spread over the surface is sometimes used, but even if rammed into the soft material, they are not likely to prove satisfactory under heavy structures, as the comparatively thin and flexible layer between the soft material and the base of the structure may not sufficiently prevent the shifting of the soft material below.

31. The effect of a deep excavation on the bearing capacity of soft and plastic soils has been referred to in Art. **25**, and very satisfactory subfoundations for ordinary structures may often be thus secured in material that, at or near the surface, would have little supporting power. Deep excavations for foundations are sometimes partly refilled with sand or gravel. This gives good results, which are probably due to the counterpressure caused by the sand or gravel on the soft material. Artificial support is sometimes provided by enclosing the area to be occupied by piles or other retaining structures. Some of these methods will be referred to later.

THEORY OF FOUNDATIONS

FOOTINGS

32. Necessity for Footings.—The office of the foundation is to form a suitable connection between a structure and the subfoundation. If, as is sometimes the case, the material of the subfoundation is equal in strength, hardness, and durability to the material of which the structure itself is built, no foundation is required in the sense in which the word is here used, except in so far as the name may be applied to the line or surface of contact between the natural material of the subfoundation and the more or less artificially prepared material of the structure. It is common to construct the lower courses of piers, walls, and like structures, with steps or projections, as shown in Fig. 10, but these projections have no utilitarian value if the material of the subfoundation is equal in strength and durability to that of the structure. Thus, in Fig. 10, if the structure is of granite or limestone and the subfoundation is of the same or equally good material, the offsets in the footing courses might safely be dispensed with, and the lines of the structure continued down to the subfoundation, as shown by the dotted lines. These projecting courses may, however, possess a certain esthetic value, which justifies their use.

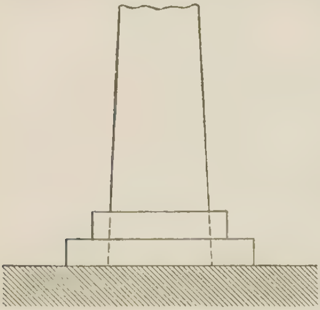
Fig. 10

Where the material of the subfoundation is inferior in strength to the material of the structure, it obviously becomes necessary to accommodate the one to the other, in order to secure equal strength throughout. The material of the structure may readily carry a load of 20 or more tons per square foot, while that of the subfoundation may be capable of carrying only 2 tons per square foot. The problem is, therefore, to distribute the load carried by the structure over such an area of the subfoundation that its safe load shall not be exceeded. This is ordinarily accomplished by the use of what is commonly called the method of **spread foundations.** Essentially, this method consists in enlarging the lower end or base of the structure so that it may cover an area of subfoundation the aggregate resistance of which will be at least equal to the weight of the structure. The usual method of accomplishing this is by stepping out or enlarging the base of the structure by offsets, called **footing courses,** or simply **footings.** The material employed in these footing courses may be the same as, or different from, that used in the body of the structure, the kind of material used being determined mainly by the conditions that are to be met in each particular case.

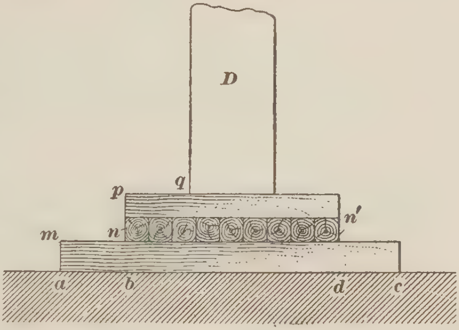

FIG. 11

33. Strength of Footings.—Footings are treated as cantilevers uniformly loaded. To secure the requisite strength most economically, two or more sets of footings are often superimposed to form steps, as shown in Fig. 11; this has the effect of decreasing their loaded free length and enabling them to sustain greater loads. In this figure, which illustrates a timber foundation, there are two footings, $m\,n$ and $p\,q$. The force acting on $m\,n$ is the upward pressure of the part $a\,b$ of the subfoundation; this pressure is assumed to be uniformly distributed, its intensity being equal to the total load on the subfoundation divided by the area of the

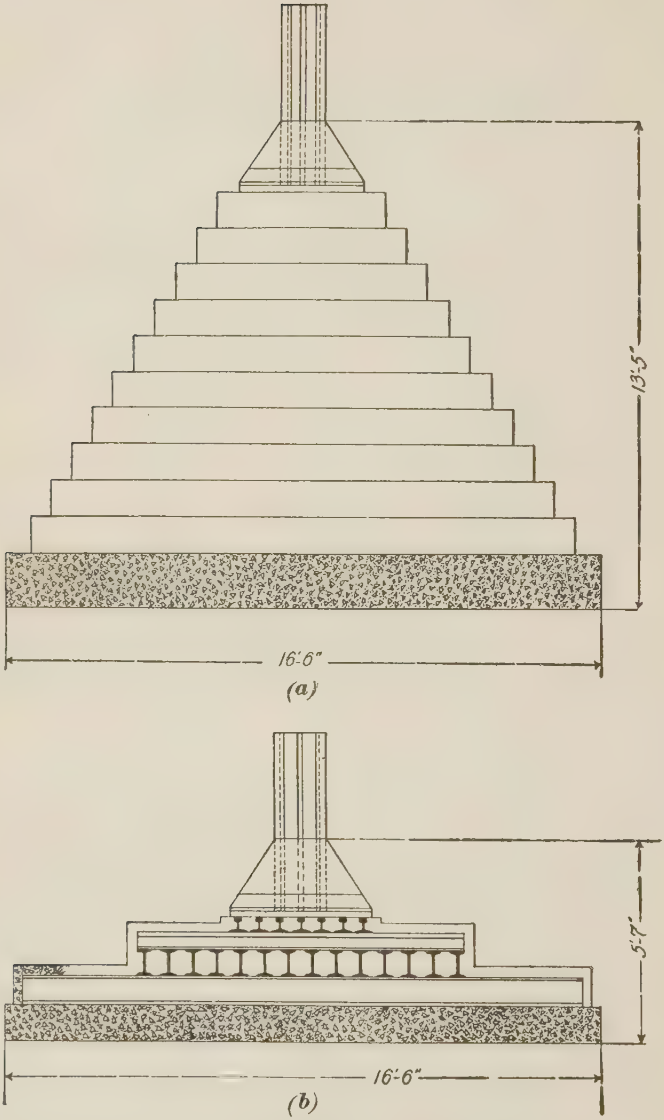

Fig. 12

subfoundation. Likewise, the force acting on pq is the upward pressure, or reaction, of the timbers nn', whose intensity is the total load on the timbers divided by the combined area of their upper or supporting surfaces.

It may be necessary to use several sets of footings; and the result will be to increase greatly the height and volume of the foundation. To obviate this increase, a stronger material, as steel, may be substituted for the beams of wood, in which case one footing may be sufficient. In masonry work, the short projections allowable with cantilevers of stone may necessitate the use of very deep and bulky foundations, and it may be desirable or necessary to substitute steel to reduce this height and bulk. Thus, in Fig. 12, (*a*) illustrates a masonry foundation for a steel column in a large building; if steel beams embedded in concrete are substituted, the foundation may be made much smaller, as shown at (*b*).

34. The formula for the load that can be safely supported by a cantilever beam is

$$W = \frac{2 s_b I}{j l c} \qquad (1)$$

in which W = total distributed load, in pounds;
c = distance from the outermost fiber to the neutral axis;
l = length of beam, in inches;
s_b = modulus of rupture, that is, ultimate bending strength, of material, in pounds per square inch;
I = moment of inertia of the section, referred to the inch;
j = factor of safety.

The value of I for a rectangular beam is $\frac{1}{12} b d^3$, and the value of c is $\frac{1}{2} d$ (see *Strength of Materials*, Part **2**). Assuming a value of unity for b, and substituting in formula **1**, these values of I and c, there results

$$W = \frac{s_b d^2}{3 j l} \qquad (2)$$

If the weight, in tons per square foot, is denoted by w, then $w = \frac{W \times 144}{2{,}000\, l}$; whence

$$W = \frac{2{,}000}{144} w\, l = \frac{125}{9} w\, l,$$

and formula **2** becomes

$$\frac{125}{9} w\, l = \frac{s_b\, d^2}{3\, j\, l};$$

whence $$w = \frac{3\, s_b\, d^2}{125\, j\, l^2} = .024 \frac{s_b}{j} \left(\frac{d}{l}\right)^2 \qquad (3)$$

From this equation follows

$$d = l \sqrt{\frac{w\, j}{.024\, s_b}} \qquad (4)$$

35. Moduli of Rupture and Factors of Safety.—For the materials most used in foundation work, the ultimate moduli of rupture and the factors of safety are commonly taken about as given in Table III. Some of these values are somewhat different from those adopted in other branches of engineering; but it should be constantly borne in mind that all such values are rough averages, more or less uncertain, and that it is this uncertainty that makes it necessary

TABLE III

MODULI OF RUPTURE AND FACTORS OF SAFETY

Material	Ultimate Modulus of Rupture S_b Pounds per Square Inch	Factor of Safety to be Used
Structural steel	60,000	4 to 8
Cast iron	20,000	9
The stronger woods, oak, Southern pine, etc.	10,000	8 to 10
Ordinary woods, white pine, hemlock, etc.	7,500	8 to 10
Granite of average good quality . . .	1,800	10 to 15
Limestone of average good quality . .	1,500	10 to 15
Sandstone of average good quality . .	1,200	10 to 15
Brick of average good quality	500	8 to 10
Concrete 28 days old (1 : 3 : 5)	250	10

to use a factor of safety. Scarcely any two investigators arrive at the same conclusion as to the exact strength of a given material; and the student should not be surprised at the differences in the values given in different books or adopted in different branches of the engineering profession.

36. Steel Footings.—In the case of rolled-steel beams (not rectangular in section), the moment of inertia of the section may be taken from such tables as are published by the manufacturers of structural shapes. For this kind of material, 15,000 pounds per square inch may be safely used as a working bending strength for foundations exposed to quiescent loads only.

In the long offsets possible with steel shapes, the deflection of the beam under this loading may be important in some structures, and should be determined by the formulas given in *Strength of Materials*, Part 2.

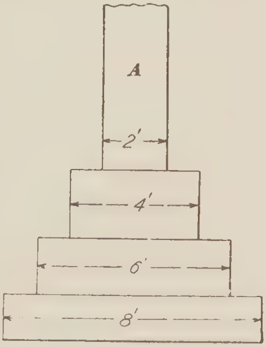

FIG. 13

EXAMPLE 1.—A wall *A*, Fig. 13, 2 feet thick, carries a load of 12 tons per lineal foot, including its own weight. The foundation of concrete is designed to have each footing project 1 foot beyond the one above. If the subfoundation may be safely loaded with 1½ tons per square foot: (*a*) what should be the width of the base? (*b*) what should be the thickness of each course?

SOLUTION.—(*a*) Since the load is 12 T. per lineal foot and the subfoundation can carry 1½ T. per sq. ft., the width of the base should be

$$12 \div 1.5 = 8 \text{ ft.} \quad \text{Ans.}$$

There will, therefore, be three footing courses. (See Fig. 13.)

(*b*) The first course below the wall has a width of 4 ft. and carries a load of 12 ÷ 4 = 3 T. per sq. ft.; the second course carries a load of 12 ÷ 6 = 2 T. per sq. ft.; and the third course, a load of 12 ÷ 8 = 1.5 T. per sq. ft. To apply formula **4**, Art. **34**, to the first footing, we have $l = 12$, $w = 3$, $j = 10$, and $s_b = 250$ (see Table III). Substituting in the formula,

$$d = 12\sqrt{\frac{3 \times 10}{.024 \times 250}} = 26.83 \text{ in.} \quad \text{Ans.}$$

For the second footing, $w = 2$, the other values being the same as above; therefore,

$$d = 12\sqrt{\frac{2 \times 10}{.024 \times 250}} = 21.91 \text{ in.} \quad \text{Ans.}$$

For the third footing, $w = 1.5$, and

$$d = 12\sqrt{\frac{1.5 \times 10}{.024 \times 250}} = 19 \text{ in., nearly.} \quad \text{Ans.}$$

Example 2.—A brick foundation is to support a pillar 2 feet square resting on a pedestal 3½ feet square and carrying a load of 150 tons. The capacity of the subfoundation is 1.5 tons per square foot. (*a*) What should be the area of the subfoundation? (*b*) If the foundation is made in four footing courses, each 1½ feet in depth, what should be the dimensions of each course?

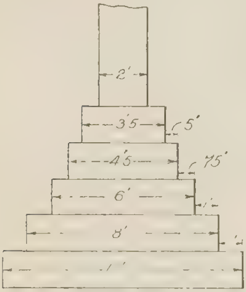

Fig. 14

Solution.—(*a*) The area of the subfoundation should be

$$150 \div 1.5 = 100 \text{ sq. ft.} \quad \text{Ans.}$$

(*b*) The base will be made 10 ft. × 10 ft. To determine the allowable length of cantilever for the first course above the subfoundation—that is, the distance the lowest course projects beyond the one above—apply formula **3**, Art. **34**. Solving for l, it is

$$l = d\sqrt{\frac{3\,s_b}{125\,j\,w}}$$

The values of s_b and j taken from Table III are 500 and 10, respectively; $w = 1.5$, and $d = 18$. Substituting in the formula,

$$l = 18\sqrt{\frac{3 \times 500}{125 \times 10 \times 1.5}} = 16 \text{ in., nearly.}$$

The lower course is made to project 1 ft.; therefore, the next course will be 8 ft. × 8 ft. The distributed load is

$$\frac{150}{8 \times 8} = 2.344 \text{ T. per sq. ft.}$$

Substituting the proper values in the formula,

$$l = 18\sqrt{\frac{3 \times 500}{125 \times 10 \times 2.344}} = 13 \text{ in., nearly}$$

This course is made to project 1 ft.; therefore, the next course will be 6 ft. × 6 ft. The distributed load is

$$\frac{150}{6 \times 6} = 4.167 \text{ T. per sq. ft.}$$

Substituting the proper values in the formula,

$$l = 18\sqrt{\frac{3 \times 500}{125 \times 10 \times 4.167}} = 10 \text{ in., nearly}$$

This course is made to project $\frac{3}{4}$ ft.; the next course is, therefore, $4\frac{1}{2}$ ft. × $4\frac{1}{2}$ ft. The stone pedestal is $3\frac{1}{2}$ ft. square; therefore, the fourth footing course projects $\frac{1}{2}$ ft.

EXAMPLES FOR PRACTICE

1. If the footing courses of the wall in Fig. 13 are composed of granite, what will be the required thickness of each, using a factor of safety of 10?

Ans. $\begin{cases}\text{Top course, 10 in.}\\ \text{Middle course, 8.16 in.}\\ \text{Bottom course, 7.06 in.}\end{cases}$

2. A brick foundation is designed to carry a load of 150 tons. (*a*) If the capacity of the subfoundation is 1.5 tons per square foot, what should be the area of the bottom course? (*b*) If the foundation is made in three footing courses, each projecting 1 foot beyond the next, what should be the depth of each of the courses?

Ans. $\begin{cases}(a)\ \text{100 sq. ft.}\\ (b) \begin{cases}\text{Top course, 22.4 in.}\\ \text{Middle course, 16.8 in.}\\ \text{Bottom course, 13.4 in.}\end{cases}\end{cases}$

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37. Special Cases.—The assumption that the footing courses in foundations may be treated as uniformly loaded cantilever beams firmly fixed at one end is not always valid, and the exceptions are so common that they must be considered. In the foundation shown in Fig. 15*a* the total load *W* from the column may be considered as distributed over the portion *c b* of the top of the beams *a*, while the upward reactions acting on the bottom of these beams, and also of the amount equal to *W*, are distributed over the entire sur-

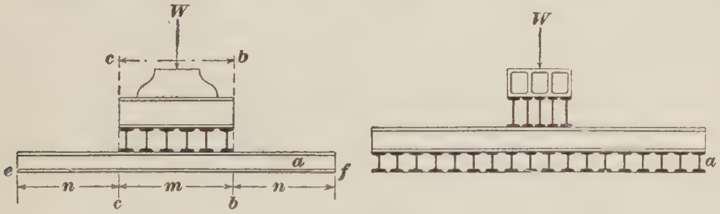
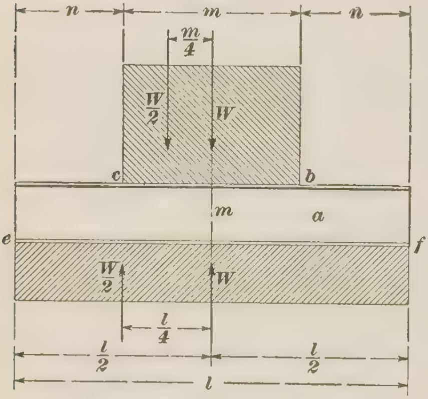


FIG. 15*a*

face $e\ f$, the bottom area $c\ b$ receiving only a portion of the load W. This loading is diagrammatically shown in Fig. 15*b*, on which each of the shaded rectangles represents the total load W, the intensity being proportional to the altitudes of these rectangles. The difference in intensity of the loading on the top and bottom of $c\ b$ gives rise to bending stresses in that portion of the beam, and it can be shown by means of advanced mathematics that the maximum bending moment occurs at a point m, midway between c and b. To find the amount of this bending moment take moments about m, Fig. 15*b*, of all the forces to the left of this section. The sum of the moments of the upward forces is equal to $\frac{W}{2} \times \frac{l}{4}$, and that of the downward forces is negative and equal to $\frac{W}{2} \times \frac{m}{4}$. The algebraic sum is



(*b*)

FIG. 15*b*

$$M = \frac{W}{2} \times \frac{l}{4} - \frac{W}{2} \times \frac{m}{4} = \frac{W}{4}\left(\frac{l}{2} - \frac{m}{2}\right)$$

or, noting that $\frac{l}{2} - \frac{m}{2} = n$, we have

$$M = \frac{Wn}{4}$$

EXAMPLE.—The total load carried by the bottom course of steel I beams, Fig. 15*a*, is 360,000 pounds; the length of the beams is 10 feet; and the width of the course next above it is 3 feet. (*a*) What is the maximum bending moment? (*b*) What size I beam may be used, assuming an extreme fiber stress of 15,000 pounds per square inch?

SOLUTION.—(*a*) The projection at each end of the bottom course is

$$\frac{10 - 3}{2} = 3\tfrac{1}{2} \text{ ft., or 42 in.}$$

There are eighteen I beams in the course; therefore, the load on each is

$$360{,}000 \div 18 = 20{,}000$$

Substituting these values in the formula, gives

$$M = \frac{20{,}000 \times 42}{4} = 210{,}000 \text{ in.-lb.} \quad \text{Ans.}$$

(*b*) Referring to a table of moments of inertia, it is found that the moment of inertia of an 8-in. I beam weighing 18 lb. per foot of length is 56.9. The resisting moment of the beam is therefore

$$\frac{15{,}000 \times 56.9}{4} = 213{,}375 \text{ in.-lb.;}$$

therefore, an 8-in. I beam may be used. Ans.

EXAMPLES FOR PRACTICE

1. If the width of the upper footing, Fig. 15*a*, is 2 feet, the loading, length, and number of beams in the course *a* being the same as in the preceding example, what will be the greatest bending moment?
Ans. 240,000 in.-lb.

2. An 8-inch I beam, weighing 18 pounds per foot, has a moment of inertia of 56.9. Assuming an extreme fiber stress of 18,000 pounds per square inch, would this beam be sufficiently strong for the load in the preceding example?
Ans. Yes.

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38. Footing Courses of Reinforced Concrete.—Footing courses of concrete reinforced with steel rods are both efficient and ecomomical. Fig. 16 illustrates a footing of this kind; it consists of a block of concrete into which are built steel rods to resist the tensile stress, the steel being placed as near as possible to the lower surface of the block.

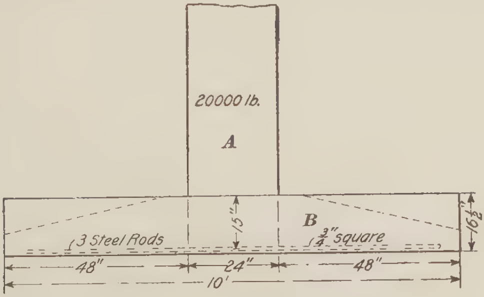

FIG. 16

In *Reinforced Concrete*, formulas and tables are included for computing the strength of reinforced-concrete beams for any combination of unit stresses of steel and concrete and for the constants and percentages of steel usually employed in practice. However, for the sake of simplicity, the values recommended by the Joint Committee for certain classes of materials and given in Arts. **69** and **70**, *Reinforced Concrete*, will here be used. Thus, the value of n, which is the ratio of the moduli of elasticity of steel and concrete, will be assumed as 15 and the allowable maximum fiber stresses of steel and concrete, as 16,000 and 650 pounds per square inch, respectively. For these conditions, the formula for the resisting moment in a reinforced-concrete beam is

$$M = Rbd^2 \qquad (1)$$

in which b is the width of the beam, in inches, and d its effective depth—that is, the distance of the center of the steel from the top of the beam—in inches. R is a coefficient depending on the amount of steel used, and its value for different percentages of steel may be taken from columns 2 and 6, Table II, *Reinforced Concrete*. M is the resisting moment of the beam in inch-pounds, and in designing the footing, the resisting moment must be made equal to the maximum bending moment—that is, to the maximum moment of the external forces. Therefore, substituting for M the value derived in Art. **37**, formula **1** becomes.

$$\frac{Wn}{4} = Rbd^2 \qquad (2)$$

Formula **2** may serve for determining any one of the five quantities involved in the equation when the other four are known. In order to use R, the value of the steel ratio p—that is, the ratio of the area of steel to that of concrete—must be known. As explained in Art. **46**, *Reinforced Concrete*, this ratio is expressed by the formula

$$p = \frac{A}{bd} \qquad (3)$$

in which A is the area of steel used in the section of the beam.

In the case shown in Fig. 16, there are assumed to be no stresses on the footing course in a direction parallel to the wall; but where structures, such as the pillar shown in Fig. 14, in which the footing course is stepped out on all sides of the load to be carried, may cause cross-stresses in the footings, two sets of steel rods placed at right angles to each other should be used.

39. Reinforced concrete may be employed to bridge over spaces between piles or other non-continuous supports. Thus, in Fig. 17, where the two rows of piles *A* and *B* are spaced far apart, and the soil between them is so soft as to afford little or no support, a plain-concrete cap *C* would be inadequate to carry the distributed load, but the cap may be made amply strong by reinforcing it with steel rods, as shown. In this case, the beam is to be treated as a beam supported at both ends, and its bending moment calculated accordingly. Its resisting moment may be found by formula **1**, Art. **38.**

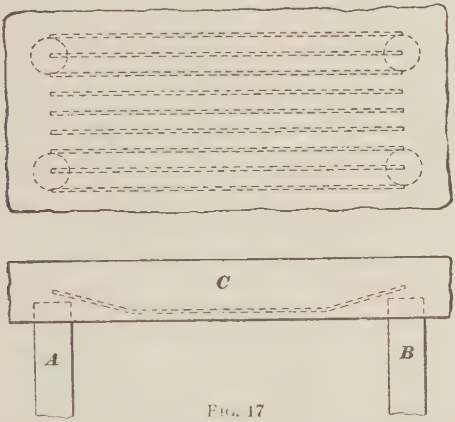

FIG. 17

EXAMPLE 1.—Let Fig. 16 represent a section of a wall *A* resting on a single footing course *B* of reinforced concrete, the dimensions being as marked, and the weight 20,000 pounds per foot of length. The steel reinforcement consists of three plain rods each $\frac{3}{4}$ inch square, evenly spaced in the 1-foot section, at a distance of 15 inches from the top of the course. To find the moment of resistance of the footing and to determine whether the beam is strong enough, using the constants recommended by the Joint Committee.

Solution.—To apply formula **1**, Art. **38**, $A = (\frac{3}{4})^2 \times 3 = 1.6875$; $d = 15$; $b = 12$. Then, by formula **3**, Art. **38**, $p = \frac{A}{bd} = \frac{1.6875}{12 \times 15} = .0094$. Referring to columns 1 and 2, Table II, *Reinforced Concrete*, the corresponding value of R is found as 114.67. Substituting known values in formula **1**, Art. **38**,

$$M = 114.67 \times 12 \times 15^2 = 309{,}600 \text{ in.-lb.} \quad \text{Ans.}$$

By the formula of Art. **37**, the bending moment is

$$M = \frac{20{,}000 \times 48}{4} = 240{,}000 \text{ in.-lb.}$$

The moment of resistance exceeds the bending moment, and the beam is, therefore, strong enough. Ans.

Example 2.—Assuming the conditions of the preceding example, let the weight of the wall be 34,000 pounds per linear foot and the steel reinforcement be placed 20 in. from the top of the footing. Find the area of steel that would be just sufficient to carry the load.

Solution.—Apply formula **2**, Art. **38**. Solving for R, it is found that

$$R = \frac{Wn}{4bd^2}$$

To apply this formula, $W = 34{,}000$; $n = 48$; $b = 12$; and $d = 20$. Substituting these values,

$$R = \frac{34{,}000 \times 48}{4 \times 12 \times 20^2} = 85$$

Referring to Table II, *Reinforced Concrete*, it is found that for the constants recommended by the Joint Committee, the value of P corresponding to $R = 85$ is .0060. Applying, now, formula **3**, Art. **38**, the required area is

$$A = pbd = .006 \times 12 \times 20 = 1.44 \text{ sq. in.} \quad \text{Ans.}$$

ECCENTRIC LOADING OF FOUNDATIONS

40. When the resultant pressure acting on a foundation does not pass through the center of gravity of the foundation, the latter is said to be **eccentrically loaded.** In the absence of external lateral forces, the center of pressure of a foundation lies in a vertical line passing through the center of gravity of the structure. If this line does not pass through the center of gravity of the foundation, the latter is eccentrically loaded. One of the most frequent examples of eccentrically loaded foundations is that of city buildings, where the wall of one building stands directly against that of its neighbor, as illustrated in Fig. 18. The footing courses can be stepped off on one side only, with the result that while the center of pressure falls at *a*, the center of gravity of the foundation is at *b*. This causes an uneven distribution of pressure over the subfoundation, and is likely to produce uneven settlement and consequent cracking of the structure. Where the subfoundation is absolutely unyielding, as solid rock, eccentric loading may generally be disregarded, unless it is sufficient to crush the material. As, however, masonry with mortar joints is likely to be somewhat compressible, it is better to avoid eccentric loading even in such cases. In the case of all structures resting on compressible subfoundations, eccentric loading should be avoided as much as possible, and, where it is not practicable to avoid it, it should be properly provided for.

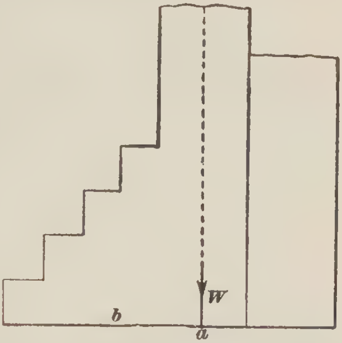

FIG. 18

MATERIALS USED FOR FOUNDATIONS

41. The materials commonly employed for foundations are masonry and timber. Steel and iron are occasionally used alone, but only under exceptional circumstances; steel rods embedded in concrete, however, are very extensively employed, reinforced concrete being now one of the materials of construction most commonly used for nearly all kinds of structures. The successful designing of foundations requires a knowledge of the properties of these materials and of the conditions and forms of construction to which they are adapted. The subject of masonry, including both plain and reinforced concrete, is fully treated in other Sections of this Course, and here it will be sufficient to make only a few remarks relating to the use of the various kinds of masonry for foundation work.

MASONRY

STONE AND BRICK MASONRY

42. Although concrete has to a great extent superseded stone and brick for foundation work, the two latter materials are still used to a very great extent. Of the several kinds of stone masonry described in *Stone and Brick Masonry*, the only ones that are commonly used for foundations are squared-stone and rubble masonry.

43. Kind and Quality of Stone Used.—The stones usually employed in masonry construction are granite, limestone, sandstone, trap rock, and occasionally massive slate. Any of these may vary in quality so widely that no stone without an established reputation for strength and durability should be used in important structures except after careful investigation. In the older settled sections of the country, the quality of stones from certain geological strata and from certain quarries has been established by long experience, and these may be used with confidence. In new countries or localities, the engineer must, in the absence of experience, determine the character of any stone by proper tests. Numerous physical and chemical tests have been proposed or devised for determining the quality of building stone, but they are often unnecessary. In many cases, strata of a rock may be observed along streams or the sides of hills, where it has been exposed for centuries to destructive climatic influences; the present condition of these strata is the surest evidence of the quality of the rock. If the exposed parts stand out boldly, with sharp angles and projections and no cracks, no better evidence of durability could be wanted; and it is also found that under such conditions the strength of the rock is usually high. Such evidence is of far greater practical value than any chemical or physical tests that can be applied. Furthermore, no physical or chemical tests should be accepted as conclusive of the value of the rock if in such natural exposures it appears cracked and disintegrated.

with distinctly rounded angles and projections, or has receded by decay under or around projecting strata of greater durability. It is true that some such rock, if quarried and properly dried or seasoned, may prove fairly durable in structures where its dryness may be maintained or where it will be more or less protected from climatic influences; but, as a rule, experience alone can safely establish such qualities, particularly in foundation work, where the stone, being exposed to moisture and alternations of heat and cold, is likely to develop any weaknesses that would appear from its natural geological exposure.

TABLE IV

Kind of Stone or Brick	Ultimate Crushing Strength		Ultimate Tensile Strength
	Pounds per Square Inch	Tons per Square Foot	Pounds per Square Inch
Limestone	6,000 to 18,000	430 to 1,300	140 to 2,500
Marble	7,000 to 20,000	500 to 1,440	150 to 2,800
Sandstone, hard	6,000 to 15,000	430 to 1,080	600 to 2,300
Sandstone, soft	4,000 to 10,000	290 to 720	150 to 1,200
Granite	10,000 to 20,000	720 to 1,440	900 to 2,700
Trap rock	15,000 to 24,000	1,080 to 1,730	
Brick, ordinary building	600 to 7,500	43 to 540	250 to 2,700
Brick, pressed	800 to 15,000	58 to 1,080	500 to 2,000
Brick, vitrified paving	8,000 to 18,000	58 to 1,300	1,500 to 3,000

44. Strength of Stone and Brick Masonry.—The engineer is most interested in the compressive resistance of stone or brick, and does not often have occasion to consider the tensile strength. The former has been determined in a large number of instances, but data on the latter are few and unsatisfactory. The resistance to crushing of the same kind of stone varies within wide limits. An idea of these variations may be obtained from Table IV. In the last column of this table is given the approximate tensile strength of the materials, according to the best available authorities. Some of the values for crushing strength have already been given in Table I. These values are given here for general information, not as the values that would be likely to be used in any actual case.

45. Cutting and Dressing Stone.—For ordinary foundation work, where the stones are laid in good Portland-cement mortar, very little cutting and dressing is required, and money expended to secure close beds or joints, or good faces, is practically wasted. Even where squared-stone masonry is used, the stone, if it comes from the quarry in reasonably good shape, needs very little dressing. It can be broken by the hammer into roughly approximate form and dimensions, and it may be necessary to use the point chisel to remove objectionable projections. Proper care to bed the masses in and to thoroughly fill the joints and all voids with good mortar will insure the requisite strength and stabililty at a far lower cost than careful and expensive dressing. This is still more true of rubble masonry, where no tool other than the hammer need be used, and that but sparingly.

TABLE V
CRUSHING STRENGTH OF MASONRY

Kind of Masonry	Crushing Strength Tons per Square Foot
Rubble, dry	3 to 10
Rubble in cement mortar	10 to 15
Squared stone in cement mortar . . .	15 to 20
Ashlar limestone in cement mortar . .	20 to 25
Ashlar granite in cement mortar . . .	25 to 30
Brick in lime mortar	3 to 5
Brick in cement mortar	5 to 12

46. Safe Loads on Masonry.—There is probably no definite or even approximately close ratio between the crushing resistance of the stone or brick of which masonry is built and that of the masonry itself. Nor is it possible, with any facilities that have been or are likely to be available, to determine the crushing resistance of large masses of masonry. In a few instances, masonry has failed under excessive loading, but in even these few cases there are not sufficiently complete data to determine the crushing resistance with any degree of accuracy. Such experience indicates that masonry of the classes mentioned in Table V may be expected to carry safely the loads stated. The lower value applies to good and the higher to the best quality of masonry of its class.

In the great majority of cases, the dimensions of a masonry structure are determined by considerations other than capacity to carry load, and the actual load imposed is so far below the ultimate strength of the masonry that this strength is not an important matter.

CONCRETE

47. Plain and Reinforced Concrete.—The subject of concrete is fully treated in *Plain Concrete* and *Reinforced Concrete*. As there stated, concrete made with Portland cement is far preferable to that made with natural cement, especially now that, owing to the development of the Portland-cement industry, this material can be obtained at comparatively low prices. In general, Portland cement is more economical than natural cement for many kinds of work; that is, for the same strength of concrete, one dollar's worth of Portland cement will make a larger volume of concrete than will one dollar's worth of natural cement.

In localities where, for any reason, natural cements are much cheaper than Portland cements, natural-cement concrete may be the more economical to use, and for many purposes its strength will be found sufficient.

For most engineering works, mortars of natural cement and sand should be in the proportion of one part of cement to two parts of sand, and concrete made of the same material should usually consist of one part of cement, two parts of sand, and four to five parts of stone. Such concrete should, at the age of 6 months, have a crushing strength of 900 to 1,000 pounds per square inch, and may be safely loaded with 8 to 10 tons per square foot.

48. Stone and Brick Masonry Combined With Concrete.—Where it is thought necessary that the outer or exposed surface of foundation masonry shall be of stone, in order to more effectively resist floating ice, heavy drift, or concussion and abrasion from passing vessels, the face of the structure may be built of any desired class of stone or brick masonry, and the interior filled with concrete, which is put in place as the masonry courses are completed. If the work is properly done, this method of construction is effective and economical. Fig. 19 illustrates, in plan, this method of construction.

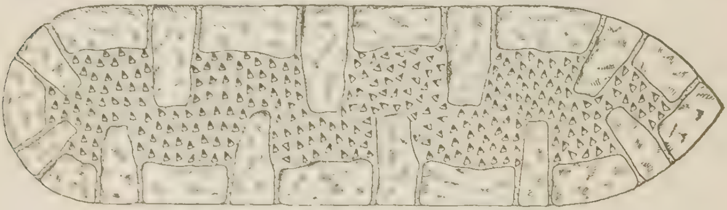

FIG. 19

TIMBER

KINDS AND PROPERTIES OF STRUCTURAL TIMBER

49. Kinds of Wood Commonly Used.—In a well-timbered country, wood is the most convenient and economical structural material. The main objections to it are that it is comparatively short-lived, and that it is easily destroyed by fire. There is a wide difference in the strength and durability, as well as the abundance and availability in different localities, of the different species of wood. Some of them, when exposed to the influences of the weather, will decay in a very short time, while others are fairly durable, even under the most unfavorable conditions.

The woods that have been most extensively employed for structural work are the following:

1. The *white oak family*, which includes common white oak, post oak, and chinkapin oak. These woods possess a

high degree of strength and hardness, and are among the most durable. Other species of oak are comparatively hard and strong, but lack durability.

2. The *pines* are important structural woods. In the northern part of the United States, white pine has long been a standard building material. It is strong, but lighter than most woods; it is soft and easily worked. The long-leaf yellow pine of the American Atlantic and Gulf States is one of the strongest, hardest, and most durable of woods, comparing favorably in these respects with white oak. On the Pacific Coast, the Oregon *red fir* is one of the best woods for structural work.

There are many species of pine that are almost equal in strength to those named, but they are short-lived. In the Southern States, loblolly pine and bastard pine are abundant, and when sent to market it requires a good knowledge of the characteristics of each to distinguish the timber from long-leaf yellow pine. In fact, a large part of the timber and lumber now on the market purporting to be long-leaf yellow pine is really loblolly or some other inferior species of pine. The engineer will require expert knowledge to be able to distinguish and reject the inferior kinds.

3. *Cypress* is a very durable wood found mostly in the swamps of the southern American states. While possessing sufficient strength, its liability to split makes it unsuitable for many structural purposes.

There are a number of other species of wood whose combined qualities of strength and durability recommend them to the engineer, but they are not sufficiently abundant to make them available for extensive structural work.

50. When, for any reason, the question of durability need not be considered, as for temporary constructions, or for foundations or parts of structures constantly submerged in water, or buried in damp clay to such a depth as to exclude the air, there are many other species of wood that possess sufficient strength and are found in more or less abundance. Among these are all the species of oak, most of the pines, hemlock, spruce, elm, ash, hickory, and others. For submerged foundation work, any of these may be utilized safely.

51. Strength and Durability of Timber.—Table VI gives the tensile and crushing strength of the species of wood commonly employed in structural work. The values given relate to sound, well-seasoned wood. In the last column, it has been attempted to designate on a scale of

TABLE VI

STRENGTH AND DURABILITY OF DIFFERENT SPECIES OF WOOD

Species of Timber	Ultimate Tensile Strength Pounds per Square Inch Parallel to Fiber	Ultimate Crushing Strength Pounds per Square Inch		Relative Durability on Scale of 1 to 10
		Parallel to Fiber	Across Fiber	
Ash, white	10,800	7,200	1,300	6
Birch, black	7,000	8,000	1,300	2
Cedar, white	6,300	5,200	500	8
Chestnut	13,000	5,300	900	6
Cypress, bald	7,900	6,000	500	9
Elm, white	10,300	6,500	1,300	4
Gum, sweet	9,500	7,100		3
Hemlock	6,500	5,300	600	2
Hickory, shagbark	16,000	9,500	2,000	5
Locust, black	18,000	9,800	1,900	10
Maple, white	10,000	6,800	1,300	2
Oak, white	13,100	8,500	1,700	8
Oak, red	11,400	7,200	1,600	5
Pine, white	7,900	5,400	600	7
Pine, long-leaf yellow	10,900	7,000	1,300	9
Pine, loblolly	10,100	6,500	1,200	3
Pine, short-leaf	9,200	5,900	1,000	3
Spruce, black	10,000	5,700	700	6
Spruce, Douglass	7,900	5,700		8

1 to 10 the relative durability of the species named, 1 representing the least durable, and 10 the most durable woods.

It must be noted, however, that the figures in this table are only approximately correct. The conditions of age, soil, climate, etc., under which the wood has grown, greatly affect both its strength and its durability. The values given in the table may be safely used with a factor of safety of about 10.

SEASONING AND PRESERVATION OF TIMBER

52. Causes of Decay.—As is generally known, the decay of wood is caused by the growth in it of fungi, many of them microscopic. These fungi require moisture and air for their development and growth. In the absence of either air or moisture, they can neither propagate nor grow. When wood is completely submerged in water, access of sufficient air to support these fungi growths is prevented, and almost any kind of wood, however short its life in open air, will remain sound for long periods of time. For this reason, species of wood that are very short-lived under ordinary conditions will prove sufficiently durable in foundations that are either always submerged in water or buried to considerable depths in soil saturated with water. Any kind of wood, therefore, that possesses the requisite strength may be safely used in submerged foundation work, provided that it is not attacked and destroyed by the Teredo or other wood-boring worms. If such woods, after thorough seasoning, could be kept perfectly free from moisture, they would prove durable even when exposed to the air. This is true of all woods, whether naturally durable or not. On the contrary, the durability of any wood will be greatly abridged if the wood is full of moisture when placed in structures in situations where the moisture will be retained, and access of air freely permitted.

53. Seasoning.—The thorough drying of wood intended for structural work is important. The process of drying is called **seasoning**, and it may be either a natural or an artificial process. Natural seasoning consists in exposing the wood to sun and air, while protecting it from rain. In artificial seasoning, the wood is subjected to artificial heat, with free access of air. The process is usually conducted in enclosed spaces called **kilns**, and the name **kiln-dried** is often applied to wood thus seasoned. The operation is greatly expedited by promoting the free circulation of air to carry away the moisture as rapidly as it is evaporated, and for this purpose power fans are often employed to keep up a constant air circulation. The process is still further hastened by first heating the air before it is forced into the kilns. Natural seasoning may require from 3 months to 1 year, depending on the conditions, while artificial seasoning may require but a few weeks. For most foundation work, where the timbers will be either submerged or subjected to constant moisture, seasoning is not necessary.

54. Chemical Preservation of Timber.—A large number of processes for preserving wood from decay and from the attacks of worms have been proposed and used more or less. They nearly all consist in filling the pores of the wood with some antiseptic that is supposed not only to destroy and prevent the growth of fungi, but to exclude moisture. Some of the chemicals used are soluble in water. Such chemicals will usually leach out when the timber becomes wet, and after a few years disappear, leaving the timber in the same condition as if it had never been treated. A few of the various processes will be very briefly described.

55. Vulcanizing.—By this process, the wood is heated in a close cylinder with air at a pressure of from 100 to 175 pounds per square inch, and a temperature of from 300° to 500° F. It is claimed that the action of the heated air evaporates all the water in the wood and coagulates the sap; that the high degree of heat sterilizes the wood and produces such changes as to render it antiseptic; and that the results are, in every other respect, very satisfactory. Ties have been treated in this way on a large scale at a cost of about 25 cents per tie.

56. Creosoting is one of the most efficient and satisfactory wood-preserving processes yet discovered. It consists in forcing into the wood the dead oil of coal tar commonly known as **creosote oil.** To properly apply the process requires an expensive plant, and increases the cost of the timber very materially. The timber to be treated is placed in a large iron vessel or cylinder, which is then closed air-tight. The cylinder is filled with live steam at a pressure of at least 50 pounds per square inch (very soft wood may require as high a pressure as 144 pounds per square inch) to heat the timber and extract the sap from it. After about 6 hours' exposure, the steam is shut off, the condensed water, sap, etc. removed from the cylinder, which is then connected with an air pump, and the air exhausted until the barometer pressure does not exceed 12 inches. The partial vacuum is maintained for a period of from 5 to 8 hours, the object being to extract the sap and moisture from the wood as completely as possible. While the partial vacuum is still maintained, the dead oil, at a temperature of about 220° F., is admitted to the cylinder until the wood is entirely covered by it. The air pump is then disconnected and a force pump connected with the cylinder, by which a pressure of at least 40 pounds per square inch is produced and maintained until the wood has absorbed the desired quantity of oil. The quantity of oil injected into the wood varies with the purpose for which the timber is to be used. For railroad cross-ties, telegraph poles, railroad trestles, etc., 10 to 12 pounds of oil per cubic foot of timber is considered sufficient; while, where the object is the protection of the wood from sea worms, from 16 to 25 pounds of oil per cubic foot should be forced into the wood. No definite figures can be given, since some woods are more porous than others and require a larger quantity of oil to properly saturate them.

57. The **creo-resinate process** consists in impregnating the wood with a compound of sixty parts of creosote, thirty-eight of rosin, and two of formaldehyde. Creo-resinate is applied in the same manner as plain creosote.

This process is the most recent, and is considered to be effective, both from an economic and from a sanitary point of view, though each of the other processes, when properly applied, will prevent decay and thus lengthen the natural life of the wood. All processes greatly diminish expansion and contraction, and also render the wood practically impermeable.

58. The **zinc-tannin process,** which is also known as the **Wellhous process,** after the inventor, consists in successively impregnating the wood with two solutions; as soon as these two solutions become mixed within the pores of the wood, a chemical reaction takes place, which turns them into a leathery substance that is insoluble in water and cannot, therefore, be readily washed out when the timber becomes wet. The chemicals employed in this process are a solution of chloride of zinc and glue, which is first forced into the wood by pressure, and a solution of tannin, which is forced into the wood after the first solution. It has been claimed that such a process, which may be applied to a cheap soft wood tie as readily as to the best oak tie, will treble the life of a soft wood tie, while it costs about 11 to 13 cents per tie. It is even claimed that the spikes in wood treated in this manner have a greater holding power than in the plain wood.

59. Other Processes.—Several other processes are employed for treating timber. One of the most common of these is **kyanizing;** this consists merely in soaking the timber in a tank containing a solution of corrosive sublimate (bichloride of mercury). The proportions used are 1 pound of sublimate to from 10 to 15 gallons of water. The process, although it has been extensively used, is objectionable in many respects, and is not effective for long periods.

Another process, known as **burnettizing,** consists in forcing into the wood, under pressure, a solution of chloride of zinc. In this process, the sap is previously removed from the timber by the method explained in Art. **56** for creosoting.

QUALITY OF TIMBER

60. Timber is likely to vary more widely in quality, and to require more careful inspection, than any other material with which the engineer has to deal. Growing trees are subject to injuries and disease that cannot be ascertained until the trees are cut into lumber. The best and strongest timber comes from thrifty live trees that have reached but not passed the age of maturity. Such trees consist of two well-defined portions, **heart wood** and **sap wood,** the latter being the outer and less matured part. Sap wood is generally not so strong and decays much more quickly than heart wood. The best of timber for structural purposes, where durability is to be considered, is free from sap wood. If, however, the timber is to be constantly submerged in water. or if it is to be treated by creosoting, the sap wood is acceptable. Structural timbers should be straight-grained, free from rotten or decayed parts, and from incipient decay, commonly called *dotiness;* from large knots, particularly if they are unsound; from cracks, whether those known as wind shakes or caused by improper seasoning; and, in short, from any imperfections that may impair their strength and durability.

STEEL AND IRON

61. Foundations are not often constructed wholly or even mainly of steel or iron, but special conditions have led to such constructions in not a few instances. Where great strength must be developed within limited spaces, or great weight must be transferred from one point to another, steel is the only adequate material. Iron and steel are used in comparatively small quantities, as for bolts and clamps in timber and masonry work; steel rods are important elements in the construction of reinforced concrete. In foundation work, where the loads are wholly static or quiescent, fiber stresses of from 14,000 to 16,000 pounds per square inch are permissible for plain steel; and in reinforced-concrete construction, where steel rods having an elastic limit of 45,000 pounds per square inch or over are used, they may be safely loaded up to 15,000 pounds per square inch of section.

62. The chief objection to the use of wrought iron and steel in foundation work is their liability to oxidize or corrode in the presence of air and moisture. Even where the metal is permanently submerged in water, the oxygen of the air contained in the water may be sufficient to cause serious oxidation, unless the metal is protected in some way. The important fact seems well established, though the reason for it is not fully understood, that iron or steel embedded in concrete, whether dry or wet, does not oxidize, and will remain for at least very long periods of time free from rust or deterioration. Combinations of steel and concrete are especially applicable to some foundation work, and, when constructed with proper care, such work may confidently be expected to prove durable and satisfactory. All steel and wrought iron used in foundations should be encased in concrete wherever possible. Cast iron is comparatively free from oxidation, even when exposed to moist air under the most unfavorable conditions.

FOUNDATIONS

(PART 2)

FOUNDATIONS ON LAND

CLASSIFICATION OF FOUNDATIONS

1. Bases of Classification.—While foundations proper may be made of a considerable variety of materials or of combinations of materials, and may differ widely in their forms and in the methods of constructing them, they may be classified with reference to either the kind of material used, or the method of construction.

2. Classification Based on Kind of Material Used. With reference to the materials employed, foundations are divided into the following classes: (1) *masonry foundations*, which consist wholly or largely of some class of masonry, such as stone, concrete, or brick; (2) *timber foundations*, which are constructed of timber, usually sawed or hewn into rectangular form (though round timber is sometimes used), framed or otherwise connected to form a solid structure or mass; (3) *pile foundations*, in which piles of either wood, metal, or concrete form the essential part of the foundation; and (4) *steel foundations*, which are made of steel.

3. Classification Based on Method of Construction. With regard to methods of construction, foundations are thus classified: (1) *dry foundations*, in which, as the name implies, no water is encountered, and no special provision for it must be made: foundations of this kind are comparatively rare; (2) *submerged foundations*, which are partly or

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wholly under water; (3) *wet foundations*, into which water flows or percolates, so that bailing or pumping is necessary; (4) *crib foundations*, which are wholly or partly composed of timber cribs or rafts; (5) *coffer-dam foundations*, in which a coffer dam is used during construction; and (6) *pneumatic foundations*, in the construction of which compressed air is used to assist the sinking of the foundation into place.

PILE FOUNDATIONS

PILES

4. Kinds of Piles.—A **pile** is a column or beam of wood, metal, or other material, driven into the earth to support or protect a structure.

Wooden piles are ordinarily the trunks of trees, freed of branches and large knots. For special purposes, however, piles sawed or hewn into rectangular sections are frequently used.

Iron and steel piles of various forms and sections are sometimes employed. If of cast iron, they are usually hollow cast tubes or columns; if of steel, they consist of single rolled shapes or of two or more such shapes riveted together to form a single member.

Concrete piles consist either of concrete reinforced with steel rods, or of plain concrete.

5. Whatever may be their form or material, piles are often classified with reference to the special purpose they are intended to serve.

Bearing piles are those used to support vertical forces or loads; to this class belong the great majority of piles used in engineering construction.

Protection piles are piles driven singly or in groups or rows to protect or shield a structure from external injury.

Anchor piles, or **mooring piles,** are used singly or in groups for holding or anchoring vessels and other floating structures.

Sheet piles consist of rows of piles, driven together as closely as possible, to enclose a foundation or other space; they are usually of rectangular section and are often tongued and grooved to insure close contact and to prevent the passage between them of water or mud.

Some special varieties of piles have received distinctive names on account of their forms or the manner of putting them in place. Among such may be mentioned *screw piles*, *jet piles*, and *pneumatic piles*. These will be described more fully in subsequent pages.

6. Use of Piles for Foundations.—Pile foundations are resorted to when the surface soil at the site of a structure is unsuitable for a subfoundation and suitable material exists only at such a distance below the surface that it is impracticable to reach and utilize it by the ordinary methods of excavation. Even if fairly satisfactory material is available, piles serve to distribute the load through a greater vertical depth than would be otherwise practicable.

THE DRIVING AND SINKING OF PILES

7. Pile Drivers.—Piles of small size to be driven to comparatively short depths may be forced into position by sledges or mauls in the hands of workmen. Larger piles are driven by special machines, called **pile drivers.** These drivers are of two kinds; namely, *drop-hammer pile drivers*, and *steam-hammer pile drivers*.

8. Drop-Hammer Pile Drivers.—Fig. 1 illustrates a very common type of drop-hammer driver. It consists essentially of a heavy weight a, called the **hammer,** and of motive power and mechanism for raising the hammer and allowing it to fall on the top of the pile. The hammer is usually of cast iron, of the form shown at (a), and weighs from 1,000 to 3,000 pounds. It is cast with vertical grooves or recesses a', a', which partly envelop and engage with the upright guides b, b, called the **leads.** These are two timbers supported on a horizontal platform, and serve to guide the hammer in its vertical motion. The leads are braced and stiffened by inclined struts *c*, *c*. A line *r* of rope or wire cable has one of its ends attached to the hammer, from which it passes over a sheave at the top of the leads and thence to the drum *d* of a hoisting engine. This drum is connected to the engine through a friction clutch, by which it may be quickly thrown into and out of gear. Another drum d_1 similarly connected to the engine, operates a second cable r_1 passing over a sheave at the top of the leads and used for hoisting the pile into place.

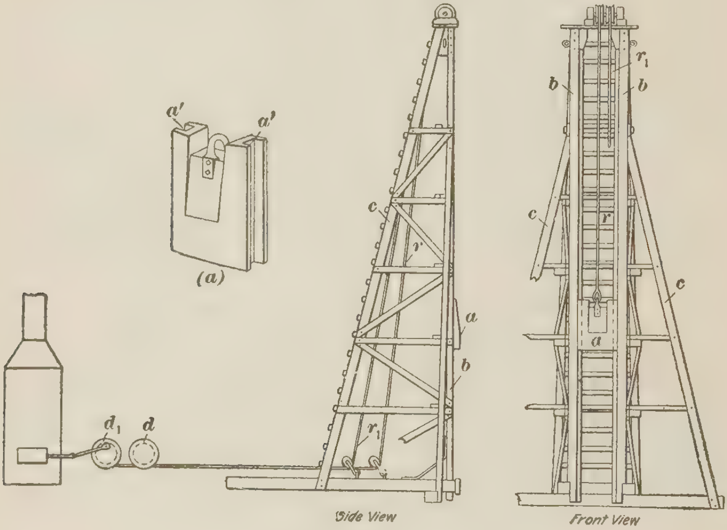

Fig. 1

In operation, the hammer is raised out of the way, and the pile is hoisted and placed upright between the leads, with its foot on the ground where it is to be driven, and secured in position by appropriate devices. The hammer is gradually lowered on the head of the pile, to force the latter into the ground to a slight depth, dependent on the softness of the soil. The hammer is then raised to any desired height, the engine stopped, and the friction clutch released, when the hammer drops on the head of the pile. In doing so, it draws the cable with it, reversing the motion of the drum, which is partly controlled by a brake that prevents it from continuing to revolve after the hammer has struck the top of the pile. This operation is repeated until the pile is driven to the desired depth.

9. The method just described of attaching the hoisting line permanently to the hammer is the one now most commonly used, and is considered the most satisfactory. Some machines are, however, still equipped with an automatic catch, called a "nipper," between the end of the line and the hammer. This device is illustrated in Fig. 2. The nippers proper *b*, *b* are attached to a cross-beam *a* that has recesses at its ends and slides up and down on the leads. The lead line is attached to an iron piece having an eye at its top, as shown. To the top of the hammer is attached a clevis having a wedge-shaped cross-piece *d* between the sides. When the nippers are lowered, they slide over this wedge-shaped piece and engage with it; and when the line is hoisted, it carries the hammer with it. At the top of the leads are placed tripping blocks, which press the curved arms together, thus releasing the hammer. The line is then lowered, the nippers again engage the hammer, and the operation is repeated. This device is seldom used now except with small drivers operated by horses.

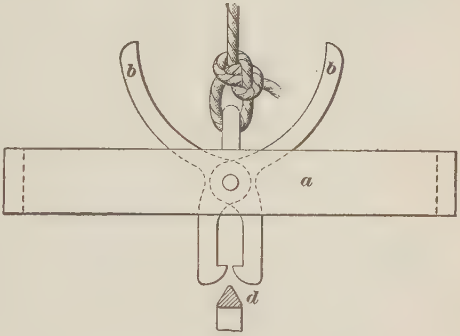

FIG. 2

10. Steam-Hammer Pile Drivers.—A steam-hammer pile driver differs from a drop-hammer machine in that the hammer is operated by steam, in the same general way as the steam hammers used for forging iron and steel. The mechanism consists of a vertical steam cylinder with a piston having a stroke of about 3 feet, the lower end of the piston rod carrying a heavy hammer. Fig. 3 illustrates the construction of the steam hammer, though the machines in actual use are more elaborate: *a* is the steam cylinder, *b* the piston rod, and *c c* the hammer, *d*, *d* are guides the upper ends of which are attached to the steam cylinder, and the lower ends to the anvil block or plate *e*. These rods pass through holes in both sides of the hammer, thus guiding its motion. The block *e* has a conical hole through its center, as shown by dotted lines, and the hammer has a projection *f* that passes freely through this opening.

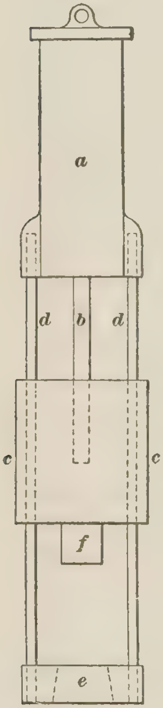

FIG. 3

Steam, conveyed to the cylinder by a flexible pipe or hose, is admitted below the piston, forcing it and the attached hammer upwards. At the end of the upward stroke, the steam is automatically cut off, the exhaust port is opened, and the hammer and piston fall by their own weight to their original position; steam is again automatically admitted, and the operation repeated. The whole mechanism is placed between and guided by the leads, and is suspended by the line that, in the drop-hammer machine, carries the hammer. When the pile is in place, the whole apparatus is lowered on its top, which is usually dressed to fit approximately the conical hole in the anvil block. Steam is then turned on and the hammer begins to work, striking rapid blows and following the pile as it descends.

11. Comparison of the Two Kinds of Pile Drivers. There has been much discussion as to the comparative merits of these two types of pile drivers. The truth is that each is most effective under conditions that favor its use. In the drop-hammer machine, the hammer may be allowed to fall any distance from 1 to 30 or more feet; while in the steam hammer the fall is limited to the stroke of the piston, usually from $2\frac{1}{2}$ to $3\frac{1}{2}$ feet. As with other falling bodies, the energy developed by the drop of the hammer is measured by the product of the weight and the distance through which it falls. With hammers of equal weight, it is obvious that the drop hammer, with its much greater possible range of fall, is capable of delivering the more energetic blows, and is therefore more effective where the material to be penetrated is very hard. On the other hand, the steam hammer delivers its blows much more rapidly than the drop hammer, and, if the material through which the piles are to be driven is comparatively soft, so that the energy of the blow is adequate to force the pile through it, more rapid progress may be made than with the drop hammer. Thus, if a steam hammer weighing 2,000 pounds and falling $3\frac{1}{2}$ feet strikes eighty blows per minute, the work done will be

$$2{,}000 \times 3\tfrac{1}{2} \times 80 = 560{,}000 \text{ foot-pounds per minute;}$$

while a drop hammer of the same weight falling 25 feet may strike ten blows per minute, and the mechanical work developed will be

$$2{,}000 \times 25 \times 10 = 500{,}000 \text{ foot-pounds per minute.}$$

In practice, the steam hammer is made from 25 to 50 per cent. heavier than the drop hammer, and this partly compensates for the smaller fall of the former. The greater energy of the blow of the drop hammer, where it is not required to force the pile through hard material, is disadvantageous, because it is much more likely to split or "broom" the head of the pile. On the other hand, the first cost of the steam-hammer outfit is much greater than that of the drop hammer, and its more complicated mechanism makes it more liable to break down and more expensive to keep in repair. The drop-hammer machine is preferred by almost all contractors.

12. Inclined Piles.—Piles are usually driven vertical. When, in exceptional circumstances, it is necessary to have them inclined, this is easily done by simply inclining the leads of the pile driver.

13. Location of Pile Drivers.—For building and other similar work, the pile-driving mechanism is installed on the ground, near the place where the piles are to be driven. When piles are to be driven in water, the pile driver is placed on a flatboat or scow, so that it will be readily movable from point to point. Pile drivers for work on operating railroads are placed on flat cars in such a manner that they can be revolved in a horizontal plane, thus allowing the leads to be swung through a considerable arc from side to side, in order to drive the piles in any position required. Drivers are also constructed so that the leads can be inclined to drive the outer or batter piles at the proper angle with the vertical.

14. Sinking Piles by Water Jet.—Where piles are to be used in sand or comparatively loose soil, they may be sunk into position by the use of a jet of water. In this method, the lower end of the pile is fitted with a nozzle discharging downwards. This nozzle is connected by hose or pipe to a force pump. When the pile is placed in position, the pump is started and a strong jet of water is discharged against the sand or other material immediately below the end of the pile. The result is to soften and cut out the material, allowing the pile to settle either by its own weight, or by additional weight placed on its top, or, sometimes, by comparatively light blows of a pile driver, until the desired depth is reached. When the operation is completed and the current of water discontinued, the surrounding material settles into close contact with the pile, giving it the necessary stability and bearing power. Under favorable conditions, piles may be driven by this method more rapidly and cheaply than with the ordinary pile driver. This is particularly true in dry sand, which offers great resistance to piles forced into it by blows from a hammer.

15. Screw Piles.—Iron or steel piles, and occasionally wooden piles, sometimes have their points provided with a horizontal spiral flange or screw, by the aid of which they are forced into sand or soft soil. Usually, one or two turns of the flange or thread are attached to the lower end of the pile, as shown in Fig. 4 (*a*) and (*b*). When such a pile is placed in position and revolved in the proper direction, the spiral blade or screw engages with the earth, forcing the pile downwards in much the same way as an ordinary wood screw is forced into soft wood. The spiral flanges vary in width, but are ordinarily from 2 to 5 feet in diameter. When once started, screw piles may be sunk pretty rapidly. The power required to turn the pile will vary with the hardness of the material to be penetrated. In soft material, the driving is accomplished by men working at levers, similar to the levers of a capstan, attached to the pile. In harder material, mechanical power is used, which is applied to the pile through suitable devices. The shaft may be made of sections of steel or cast iron connected by screws or flanges, as shown in the figure.

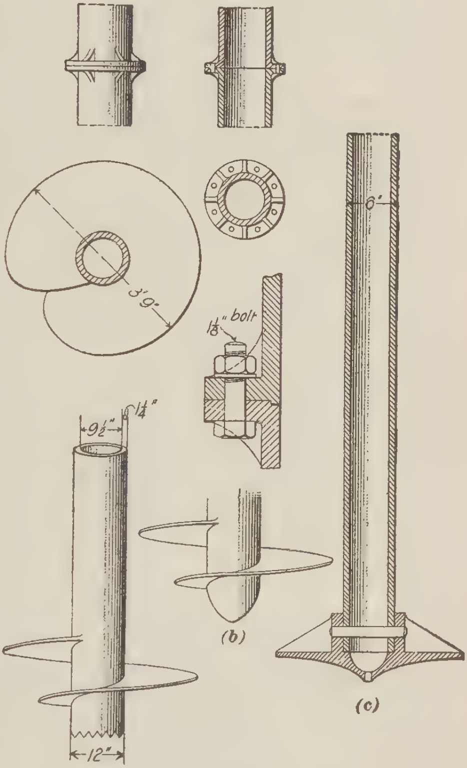

FIG. 4

One advantage of this form of pile is that the wide flange greatly increases the bearing area of the pile in soft material, where the ordinary pile might not be effective. Screw piles are frequently used in preparing foundations under water or in soft boggy locations.

CONCRETE PILES

16. Concrete piles are used to a considerable extent in foundation work. They possess the great advantage that they do not decay like wood nor corrode like steel piles. Two types will be described here.

A tube or shell of thin steel, shaped like the intended pile, is constructed so that it is partly collapsible along one or more longitudinal lines or points. This shell, which is called a **pilot pile,** is driven into the ground in much the same manner as the ordinary wooden pile. When driven, it is partly collapsed and withdrawn, and the hole made by it is filled with concrete. Where the ground is soft and the walls will not maintain their form after the pilot pile is withdrawn, a thin casing of metal or other material fitted over the pilot pile is driven with it; and, when the latter is withdrawn, the casing preserves the walls of the hole until it is filled with concrete. This kind of concrete pile has been very successfully used for the foundations of heavy buildings.

In another form of concrete pile, the pile is made of concrete in molds of the desired shape and dimensions, and allowed to set or harden. It is then driven by the water-jet process or by careful driving with a pile driver. These piles are usually reinforced with steel rods or bars, and are thus made sufficiently strong to be readily handled and driven. They are also usually provided with iron or steel shoes, to prevent injury to their points and to facilitate driving. If hammer-driven, their tops are protected with an elastic cushion of wood or other material.

SUPPORTING POWER OF HAMMER-DRIVEN PILES

17. Theoretical Formula.—Many efforts have been made by engineers to derive a satisfactory formula for the bearing capacity of piles. These formulas are usually derived from a theoretical analysis of all the conditions involved. The resistance of the ground is determined by the penetration of the pile during the last few blows of the pile driver.

Experience has proved that it is practically impossible to derive a purely rational formula, on account of the uncertainty of the conditions. It has been found that the only reliable practical formulas are those that, although having a theoretical basis, have been modified according to the results of experience.

Let R = resistance or bearing capacity of a pile;
s = set of pile, or distance that pile is driven during last blow;
w = weight of pile hammer;
h = fall of hammer during last blow.

Then, according to the principles of mechanics,

$$Rs = wh;$$

and, therefore,

$$R = wh \div s$$

This is the basis of all theoretical formulas. In this formula, R and w are in the same units, such as pounds, tons, kilograms; also, h must be in the same units as s, whether feet, inches, centimeters, etc. If the set s of the pile under the last blow is expressed in inches, and the fall h of the hammer in feet, the formula becomes

$$R = 12\,wh \div s$$

If a factor of safety of 6 is adopted, then

$$R = 2\,wh \div s$$

18. Engineering News Empirical Formula.—On account of the extra resistance caused by the settling of the earth around the pile between successive blows, the divisor in the last formula of the preceding article has been arbitrarily increased from s to $s + 1$ for drop-hammer drivers, and to $s + .1$ for steam-hammer drivers, and the following practical formulas have been proposed:

For drop-hammer pile drivers,

$$R = \frac{2\,wh}{s + 1} \qquad (1)$$

For steam-hammer pile drivers,

$$R = \frac{2\,wh}{s + .1} \qquad (2)$$

Formula 1 is called the **Engineering News formula**, because it was first published by that engineering journal.

It has been very extensively adopted, as experience has proved that it is as reliable as any formula can justifiably claim to be. The uncertainties of pile driving are so great that it is useless to attempt to use a more accurate formula.

EXAMPLE 1.—A pile was driven with an ordinary hammer weighing 2,400 pounds. The sinking under the last five blows was 22 inches. The fall of the hammer during the last blows averaged 28 feet. What was the safe bearing power of the pile?

SOLUTION.—Here the value of s may be taken as the average of the total sinking during the last five blows, or $22 \div 5 = 4.4$ in. Then, $w = 2{,}400$ lb.; $h = 28$; and $s = 4.4$. Substituting these values in formula **1**,

$$R = \frac{2 \times 2{,}400 \times 28}{4.4 + 1} = 24{,}889 \text{ lb. Ans.}$$

EXAMPLE 2.—Piles are to be driven by a steam pile driver. The hammer weighs 5,500 pounds, and its fall is 40 inches. It is required that the piles shall be driven until each shall have a safe bearing value of 30,000 pounds. What must be the sinking under the final blow?

SOLUTION.—In this case R is given, the unknown quantity being s. From formula **2** the following is derived:

$$s = \frac{2\,w\,h}{R} - .1$$

In this case, $R = 30{,}000$; $w = 5{,}500$; and $h = 40$ in. $= 3.33$ ft. Therefore,

$$s = \frac{36{,}630}{30{,}000} - .1 = 1.2 - .1 = 1.1 \text{ in.}$$

Hence, the piles should be driven until their penetration for each blow is approximately 1 in. Ans.

EXAMPLES FOR PRACTICE

1. A pile is driven by a drop-hammer weighing 2,500 pounds, with a fall of 16 feet. The last blow of the hammer drives the pile 1 inch. What load will the pile carry safely, according to the Engineering News formula?
Ans. 20 tons, nearly

2. It is desired that a pile shall be driven with a drop hammer, weighing 3,000 pounds and having a fall of 15 feet, until it shall be able to carry safely a load of 24 tons. What, by the Engineering News formula, must be the penetration at the last blow of the hammer?
Ans. .9 in., nearly

3. A pile, driven with a steam hammer weighing 5,000 pounds and having a fall of 42 inches, can safely carry a load of 32,000 pounds. What is the penetration at the last blow of the hammer?
Ans. 1 in., nearly

MISCELLANEOUS MATTERS RELATING TO PILE CONSTRUCTION

19. Pile Ends.—The lower end of a pile is usually, though not always, dressed down to a roughly conical point. Where wooden piles are driven in very hard material, or in gravel and stony ground, their points are liable to be battered and injured. To protect the point, it is often provided with an iron shoe. Several forms of shoes are shown in Fig. 5 (*a*), (*b*), and (*c*). It is also necessary or desirable to protect the heads of the piles, particularly when the drop hammer is employed and the fall is great. The most common device for this purpose is a heavy iron ring surrounding the head of the pile, as shown in Fig. 5 (*d*). The wood may be dressed down to receive the ring, but more commonly the ring is adjusted on the flat top of the pile and the hammer allowed to fall on it, forcing the ring into the wood. It is generally considered more desirable to use a heavy hammer with a low fall than a light hammer with a high fall. With high falls, the danger of splitting or breaking the pile or of unduly battering and brooming its head is greatly increased.

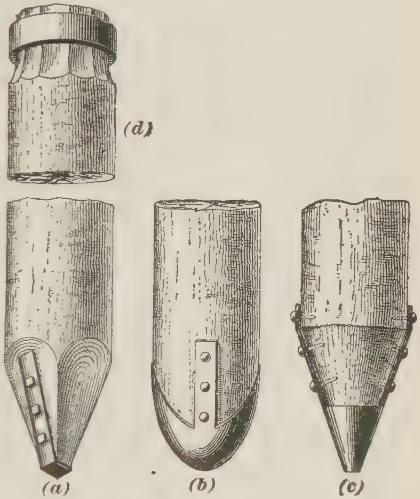

Fig. 5

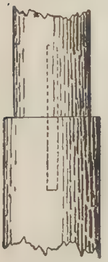

Fig. 6

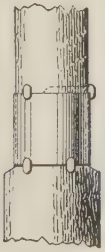

Fig. 7

20. Splicing Piles.—When a firm subsoil cannot be obtained except at a very great depth below the surface, so that it becomes impracticable, if not impossible, to obtain timber piles of the requisite length, it may be necessary to splice the piles. In general, a **splice** is some form of joint connecting the ends of two piles. It may be a dowel or iron bar about 2 feet long, embedded about 1 foot in each pile, as illustrated in Fig. 6. A still more secure joint is a band of iron, at least 12 inches long and of as great a diameter as necessary, to which the ends of both piles are fitted. To prevent the possibility of the band being jarred either up or down, away from the joint, spikes are driven into the piles immediately above and below the band, as illustrated in Fig. 7.

When the ground is in a partly fluid state, it may have so little stiffness that the jointed pile is likely to buckle at the joint, unless the splice is made in the form of scarfs, which consist of six or eight timbers about 3 inches square in sectional area and 8 to 10 feet long, spiked to the piles, as shown in Fig. 8. Whatever form of splice is used, the first or lower pile is driven into the ground until its top is but a few feet above the surface; then the second pile is spliced on and driven in. Of course, the force required for driving in a soil so soft that so long piles are necessary is correspondingly small.

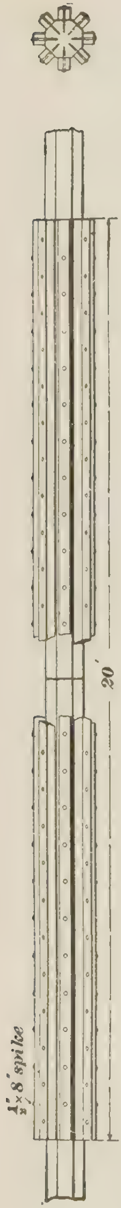

Fig. 8

21. Cutting Off the Piles.—When a timber foundation is to be constructed on piles, it is necessary to cut off the tops of the piles to bring them to an even horizontal plane, so they may properly receive the caps or timbers that rest directly on them. When not submerged, the piles may be cut off by hand, but it is usually more expeditious and economical to do this by mechanical power. A circular saw at the end of a vertical shaft is commonly employed, the frame that carries the shaft and saw being made adjustable in height and attached either to the leads of the pile driver or to some part of the platform carrying the pile driver. The power to operate the saw may be supplied by the hoisting engine of the pile driver. Where a concrete foundation is to be used in connection with the piles, it may not be necessary to cut off the piles, provided that they stand at approximately the same height and their heads are not so much broomed as to prevent a firm contact between them and the surrounding concrete; in all important foundations, however, it is generally advisable to cut off the piles.

22. Amount of Driving Necessary.—While it is important that piles should be driven to a firm seat and bearing in the soil, it is not advisable to overdrive them. It is not thought advantageous to continue hammering a pile after it has ceased to move about ½ inch with each blow, and many engineers consider a pile sufficiently driven when it fails to move more than 1 inch with each blow. Piles driven until they will go no farther, or until they move but a small fraction of an inch per blow, are said to be "driven to refusal."

When piles are driven through clay to an underlying rock floor, driving should cease as soon as the point of the pile rests solidly on the rock; further hammering may batter the point of the pile and lessen its supporting power.

23. Spacing of Piles.—When a large number of piles is required, it is best not to attempt to drive them nearer together than about 2½ feet from center to center, or about one pile to each 6 square feet of ground surface; if driven more closely, the great compression of the soil may tend to force upwards those already driven. The danger of this may be lessened if the center piles of the group are driven first and the work proceeds outwards from them.

24. Followers.—When piles are located in water and their heads must be driven below its surface, a short pile or timber, called a **follower**, is set up on top of the pile when its head reaches the surface of the water, and the driving continued until the pile is sufficiently driven.

25. Withdrawing Piles.—It is sometimes necessary to pull or withdraw piles already driven, particularly where they have been used for the construction of coffer dams and similar work. Various devices have been used for the purpose. For short sheet piling, a powerful lever worked by hand is often sufficient; when this is ineffective, powerful block and tackle operated by a hoisting engine or other power is employed. The force required to withdraw a pile is about the same as the ultimate load the pile will support.

26. Timber Suitable for Piles.—Any timber strong enough to withstand driving will have sufficient strength for foundation purposes, and if the piles when in place are below the surface of the water or are deeply submerged in damp earth, they will prove sufficiently durable. But if their tops are to be exposed to the air or submerged in water but a part of the time, it is necessary to select species of timber that are not subject to rapid decay. It is often advisable to treat the timber chemically. For not very hard soils, spruce and hemlock make good pile material; for harder soils, the hard oaks, pines, and elm may be used.

SHEET PILING

27. While sheet piling is sometimes employed for carrying loads temporarily, its use primarily is to form an enclosure around a foundation area to shut out water, silt, etc. For this purpose, sawed timbers or plank are nearly always used. When strength to support great pressure is not required, ordinary plank from 3 to 4 inches thick will be sufficient. When greater strength is required, square timbers may be used, but more frequently two or more thicknesses of plank are employed. The purpose for which sheet piling is used makes it important that the joints between adjoining piles should be as close and as nearly water-tight as possible. Several devices for securing this condition are in use, a number of them being shown in Fig. 9. In this figure, (*a*) represents the plain sheet piling, which depends for close joints on the care and accuracy with which it is driven. In (*b*) are shown two contiguous rows of piling, one row breaking joint with the other; three rows of plank are sometimes used, the outside rows being of thinner plank than the central or main row. In (*c*), (*d*), (*e*), (*f*), and (*g*) are shown various forms of "tongued-and-grooved" sheet piling; the object of this construction is not only to make a tight joint, but also to assist in guiding the piles truly in the same plane. The form (*d*) is very frequently used, the tongue and groove being formed by spiking narrow strips on the edge of the pile. The form (*g*) is a patented form known as the "Wakefield" pile; its construction is evident from the figure without further description. The points of sheet piles are usually sharpened to facilitate driving, and one side is often sloped, as shown in (*h*), to cause the piles to come into close contact with one another.

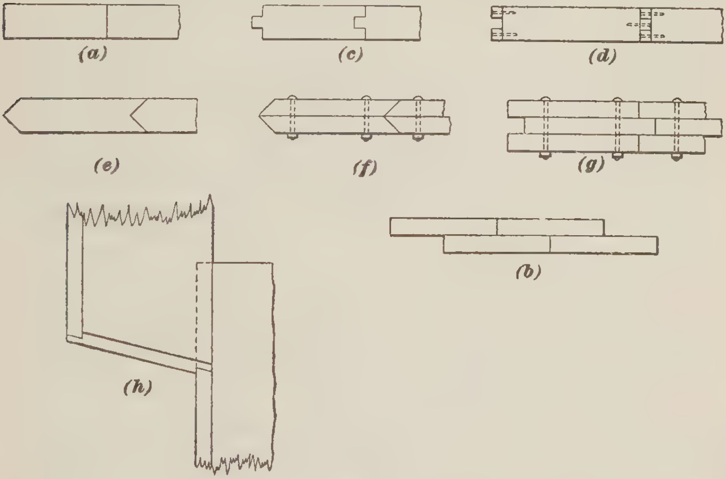

FIG. 9

CONSTRUCTION OF PILE FOUNDATIONS

28. When land foundations are to be carried to great depths to reach solid rock or other strong subfoundation, various expedients are in use to avoid the excavation of deep continuous trenches, which are expensive and often troublesome. The simplest expedient is a pile foundation. The depth to which the piles are driven depends on the character of the soil and the weight to be carried by the piles. This may best be determined by the experimental driving of a few piles, though test pits or borings will generally indicate pretty closely the probable length required. The true criterion is the resistance of the piles to movement with the last few blows of the hammer (see Art. 18). A pile foundation is really a subfoundation, which must be connected to the structure by a foundation proper. Two principal methods are used for this purpose; namely, by *timber grillage*, and by *concrete filling*.

29. Timber Grillage.—A **grillage** consists of two or more courses of timber laid at right angles to each other, each course consisting of parallel timbers laid at intervals usually not much greater than the width of the timber. Grillage is often used as a foundation on a soft-soil subfoundation or on piles. When used with piles, the latter, properly spaced, are driven in rows, as many rows being placed as are required to cover properly the foundation area.

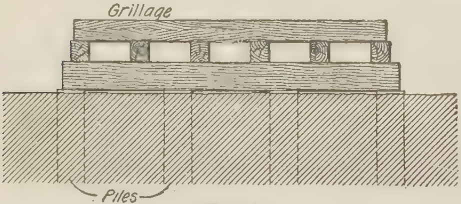

FIG. 10

The tops of the piles are cut off to a horizontal plane as near the earth as practicable, and a line of timbers, called **caps**, are placed on the tops of the piles along each row, and are secured to them by drift bolts.

It is necessary to exercise care to have the piles truly in line in the rows, in order that the caps may have a full bearing on the tops of the piles. On the caps, and at right angles to them, another set of timbers, spaced from 1 to 3 feet apart, is placed and drift-bolted to the caps. Several such courses of timbers may be used if desirable, each being placed at right angles to the previous course. This construction is illustrated in Fig. 10.

30. Concrete Filling.—In concrete filling, it is not so essential that the piles be driven accurately in rows, nor that their tops be cut off, if they are all nearly at the same level as driven; although, as the tops are likely to be more or less battered or broomed by driving, a better connection between them and the concrete will be secured by cutting them off, or at least by removing the broomed part with axes or adzes. This being done, the loose or soft portions of the earth are removed so as to give the concrete a good bearing on it, and the concrete is deposited between and around the piles and carried to a height of at least 1 foot above their tops, where it is formed into a level surface for the reception of the masonry.

ORDINARY MASONRY FOUNDATIONS

31. General Principles and Methods.—The construction of **ordinary masonry foundations**, consisting of superimposed courses of masonry either with or without footings, does not present any special difficulty. The required area of subfoundation, and the dimensions of the footings, if any, are computed by the principles explained in *Foundations*, Part 1.

32. Excavation: Sheeting.—In many cases, foundations are carried to a depth just sufficient to secure firm material, or to be below the disturbing action of frost. Frequently, however, the subfoundation material near the surface is unsuitable, or the unusually heavy loads to be carried necessitate going down to a strong stratum of material.

Very deep excavations, even in soils that are tolerably dry and firm, are liable to cave in, unless made with very flat slopes, or unless the banks are sustained by artificial supports. The means of support most commonly employed is called **sheeting.** It consists of a lining of plank held in place by longitudinal stringers, and cross-struts that brace one side of the excavation against the other. The method of excavating by sheeting is illustrated in Fig. 11, where *a*, *a* represent the planking, *b*, *b* the longitudinal stringers, and *c*, *c* the struts. Between the ends of the struts and the stringers, wooden wedges are driven to hold the planking tightly against the earth wall, or one end of the struts is fitted with a jack-screw by which any desired degree of pressure can be applied.

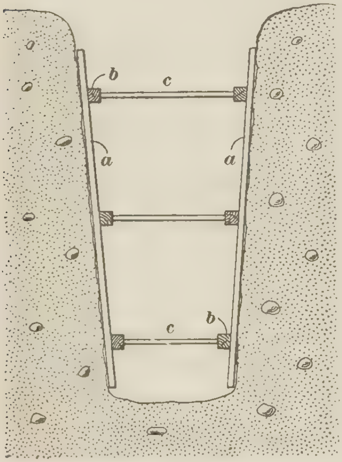

Fig. 11

33. Base Course.—The most satisfactory base course for a land foundation is a bed of concrete. It is comparatively economical; the mortar when placed accommodates itself to any irregularities of the subfoundation; and, when hardened into a monolithic mass, the bed has sufficient strength to distribute the loads properly and to bridge over any soft places in the subfoundation. If the excavated trench is cut to accurate dimensions, no forms or molds are necessary to hold the mortar in place.

34. Removal of Water.—In all land foundations of considerable depth, and in shallow ones in swampy soils, more or less water is likely to be encountered, and must be drained or otherwise disposed of. Frequently, the quantity is so small that it may be removed by ordinary buckets, which were probably the earliest devices for the purpose. For this reason, doubtless, the removal of water from such foundations is called "bailing and pumping." The cost of removing water often constitutes a large percentage of the whole cost of the foundation. Where the quantity of water is large and it must be raised to a considerable height, power pumps are employed.

TUBULAR FOUNDATIONS

35. **Tubular foundations,** called also **curb foundations,** are frequently used. They consist of a tube or curb of any desired size, either cylindrical or square, built of steel, cast iron, or wood. The method of sinking the tube is illustrated in Fig. 12, where *A* represents a cylinder built of sheet steel. A well, somewhat larger in diameter than the tube, is first excavated to as great a depth as possible without support for the walls. The cylinder is then placed in the well, and the excavation continued inside it, the earth being removed from under its edge, as shown, so that the cylinder sinks either by its own weight or by loads placed on its top, following the excavation downwards to the required depth; additional lengths are riveted to the top as necessary. The friction of the outer surface of the cylinder against the earth walls increases, of course, with the depth, and may limit the depth attainable. This friction varies from 100 to 150 pounds per square foot of surface, depending on the depth and the coefficient of friction between the cylinder and the material through which it is sunk. When the friction becomes so great that the cylinder can be forced downwards no farther, a second cylinder, smaller in diameter than the first, may be started in the bottom of the excavation and sunk in the same way as the first. Any number of additional cylinders may be thus used, and the excavation carried to any desired depth, provided that the first cylinder is made of sufficient diameter.

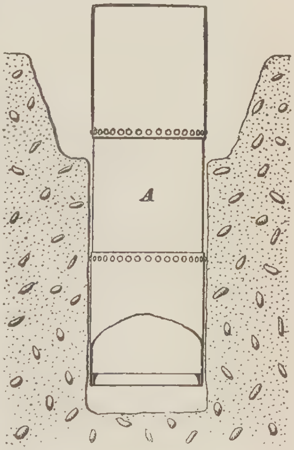

FIG. 12

36. If too much water is encountered to be conveniently handled by pumps, some form of dredge bucket may be used to carry on the excavation. After the tube has reached the bed rock or other satisfactory bearing material, it is filled with concrete or brick masonry, and a suitable bearing cap is placed on its top. A sufficient number of these tubes being in place to support the proposed structure, steel beams are placed over them, or they are connected by brick or concrete arches. Where water is encountered, these tubes may be fitted with air locks and sunk by the pneumatic process, as will be described further on. Wooden cylinders lined with brick were the earliest representatives of this class of work, and have been sunk to very considerable depths.

FOUNDATIONS FOR BUILDINGS

37. General Considerations.—The construction of adequate foundations for the great modern buildings of cities presents difficult and peculiar problems, which have developed a special class of foundation work. Large cities are built on geological formations differing widely in their physical characteristics, and, as might be expected, there has been developed in each a general method of foundation construction best adapted to its peculiar conditions. The characteristic foundation in Chicago, for instance, is different from that in New York. Certain general conditions and principles are, however, common to all, and a brief consideration of these is all that can be given here.

38. The restricted area and the great cost of city real estate in the business districts make it necessary that every available square foot of each building lot be enclosed and utilized for business purposes. The walls of the building therefore extend fully to the boundary lines, and their foundations can be extended only inwards. Even in that direction, basement and cellar space is so valuable that it may be given up for foundation purposes to such extent only as is absolutely unavoidable. This scarcity of space and the enormous cost of land have led to the construction of exceedingly high buildings, and this in turn has resulted in concentrated foundation loads heretofore unprecedented. At the same time, the walls of buildings are frail compared with most other engineering structures, and any uneven settlement of the foundation is liable to result in cracks and disfigurements.

39. Subfoundation.—To support the enormous weights that must be carried, the first and most obvious expedient has been to carry the foundation down to solid rock or to a stratum of hard pan or compacted gravel having the necessary bearing resistance. Where this is not practicable, and a compressible subfoundation must be used, the aim has been to spread the foundation over an ample area, and to so distribute it with reference to the varying weights of different parts that the pressure per unit area will be as nearly as possible the same. To avoid eccentric loading of the foundations, various devices have been used, some of which will be described presently.

In cities where compressible subfoundations are used, the foundations are placed at a sufficient depth to be below the basement, and are spread over a large area. Great care is exercised to make the intensity of pressure the same under every part of the foundation.

CANTILEVER FOUNDATIONS

40. To obviate eccentric loading, and to transfer loads from points where support is not practicable to other points, steel beams, especially cantilevers, are extensively employed. An example of their use is illustrated in Fig. 13, which represents the wall of a new steel-frame building placed directly against the wall *b* of an old building. If the column *f* rested directly on the foundation *a*, it would produce eccentric loading. To provide against this eccentric loading, the cantilever beam *d* is introduced, which transfers the center of pressure from *c* to *e*, directly over the center of the foundation. This example may be regarded as typical of most cases of the employment of cantilever beams in foundation work, and its conditions may be briefly investigated.

Let W = weight on column *f*;
R_1 = reaction of foundation at *e*;
R_2 = reaction of column *h*, to which the cantilever *d* is riveted.

The meaning of the letters *l* and *m* is shown in the figure. Since the cantilever is in equilibrium under the action of the forces W, R_1, and R_2, we have, taking moments about the left-hand end of the cantilever,

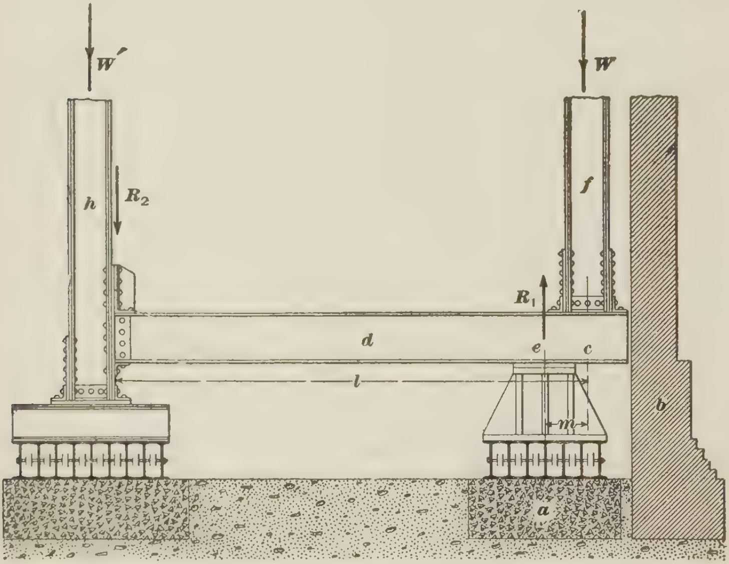

FIG. 13

$$Wl = R_1(l - m);$$

whence $$R_1 = \frac{Wl}{l - m} \quad (1)$$

Taking moments about *e*,

$$Wm = R_2(l - m);$$

whence $$R_2 = \frac{Wm}{l - m} \quad (2)$$

The cantilever exerts on the column *h* an upward force equal to R_2, and diminishes the pressure of the column *h* on its foundation. It may occur that R_2 is greater than the weight W' on column h, in which case the resultant pressures on the foundation under that column will be negative, and the column must be anchored to its foundation to prevent its being lifted from its bearing.

CONTINUOUS FOUNDATION UNDER TWO COLUMNS

41. While it is desirable, with compressible subfoundations, that each pier or column should have its independent foundation, it may happen that two or more such piers or columns located near each other must be placed on the same foundation. In such a case, the important requisite is that the resultant of the downward pressure of the two columns shall coincide with the center of gravity of the subfoundation. If the weights on the columns are different, it is important also that the foundation shall be so designed as to secure an equitable distribution of the stresses throughout it. These conditions cannot always be secured, but may be approximated.

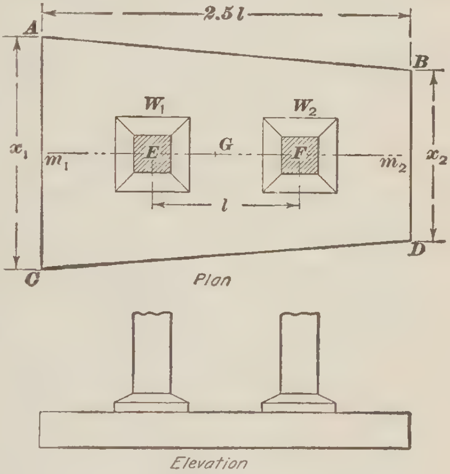

Fig. 14

With two columns carrying different weights, a trapezoidal form of foundation is naturally suggested. Such a foundation may be designed by the following method:

In Fig. 14, let $A B D C$ represent a trapezoidal foundation carrying the two columns E and F, the distance between their centers being l. Let the weights carried by the columns E and F be W_1 and W_2, respectively, W_1 being the larger. The total weight on the foundation being $W_1 + W_2$, if the subfoundation may be safely loaded with s tons per square foot, the required area A will be $\frac{W_1 + W_2}{s}$. The length of the foundation should preferably be not less than $2\,l$ nor more than $3\,l$. The widths of the ends of the trapezoid may be made proportional to the loads W_1 and W_2. If the width at the wide end is denoted by x_1 and that at the other end by x_2, then,

$$\frac{x_1}{x_2} = \frac{W_1}{W_2};$$

whence

$$x_2 = \frac{W_2 x_1}{W_1}$$

If the whole length (altitude of the trapezoid) of the subfoundation is made $2.5\,l$, then,

$$A = \frac{2.5\,l}{2}(x_1 + x_2) = 1.25\,l\left(x_1 + \frac{W_2 x_1}{W_1}\right)$$
$$= 1.25\,l\,x_1\left(\frac{W_1 + W_2}{W_1}\right);$$

whence

$$x_1 = \frac{A\,W_1}{1.25\,l(W_1 + W_2)}$$

Or, since $A = \frac{W_1 + W_2}{s}$,

$$x_1 = \frac{W_1}{1.25\,l\,s} = \frac{4\,W_1}{5\,l\,s} \tag{1}$$

Also,

$$x_2 = \frac{W_2 x_1}{W_1} = \frac{4\,W_2}{5\,l\,s} \tag{2}$$

The center of gravity G is determined by one of the methods explained in *Analytic Statics*, Part 2.

Let the distances $E\,G$ and $F\,G$ be denoted by y_1 and y_2, respectively. In order that the resultant of W_1 and W_2, which is $W_1 + W_2$, may pass through G, we must have, taking moments about E,

$$(W_1 + W_2)\,y_1 = W_2\,l;$$

whence

$$y_1 = \frac{W_2\,l}{W_1 + W_2} \tag{3}$$

Similarly,

$$y_2 = l - y_1 = \frac{W_1\,l}{W_1 + W_2} \tag{4}$$

The distances $m_1 E$ and $m_2 F$ can now be easily computed, located, and staked out.

EXAMPLE.—Suppose that in Fig. 14 the distance between the centers of the columns E and F is 8 feet, and the loads are 200 and 150 tons, respectively. If the length of the trapezoid is 20 feet and the capacity of the subfoundation 1.5 tons per square foot, what is the distance $E m_1$?

SOLUTION.—To apply formulas **1** and **2**, we have $W_1 = 200$, $W_2 = 150$, $l = 8$, and $s = 1.5$. Substituting in the formulas,

$$x_1 = \frac{4 \times 200}{5 \times 8 \times 1.5} = 13.33 \text{ ft.}$$

$$x_2 = \frac{4 \times 150}{5 \times 8 \times 1.5} = 10 \text{ ft.}$$

It is found that the center of gravity of the trapezoid is 9.5 ft. from $A C$.

The distance of E from the center of gravity of the trapezoid is found by formula **3**:

$$y_1 = \frac{150 \times 8}{200 + 150} = 3.43 \text{ ft.}$$

Therefore, $E m_1 = 9.5 - 3.43 = 6.07$ ft. Ans.

FOUNDATIONS UNDER WATER

42. Where the foundation of a structure must be placed in water, one of two general methods is followed: either the space to be occupied is freed of water, after which the work proceeds as on land, or some device for sinking the foundation through the water is employed.

COFFER-DAM PROCESS

43. Coffer Dams.—A **coffer dam** is a water-tight structure enclosing a space from which the water may be pumped out, leaving a comparatively dry area on which a foundation may be constructed as on dry land. A great variety of coffer dams have been devised and used. The simplest and most primitive, suitable for use in very shallow water, is a bank of earth, clay, or sand, rising above the surface of the water, surrounding the area to be built on.

Next in order is a simple timber structure surrounded by a bank of earth. Fig. 15 shows a cross-section of a coffer dam of the latter form. A wall of sheet piling is driven around the space to be unwatered, the tops of the sheet piles are spiked to a timber, called a **wale** or **waling piece,** and a bank of earth is then deposited around the outside of the structure.

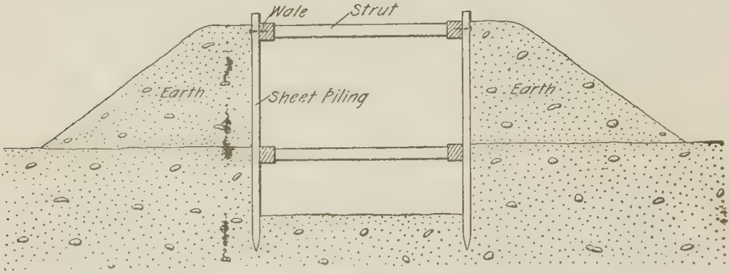

FIG. 15

44. A form of coffer dam very frequently used is illustrated in Fig. 16. It consists of an inner and an outer line of sheet piling with waling pieces at the top, tied together with iron bolts. Frequently, each wall is composed of two or more lines of sheet piling, as shown on the left-hand side, the individual piles breaking joint for the purpose of increasing the strength of the structure, and of making the walls themselves as nearly water-tight as possible. The space between the two walls is then filled with earth or other suitable material to exclude the passage of water. This material is called **puddle.** The water may then be pumped from

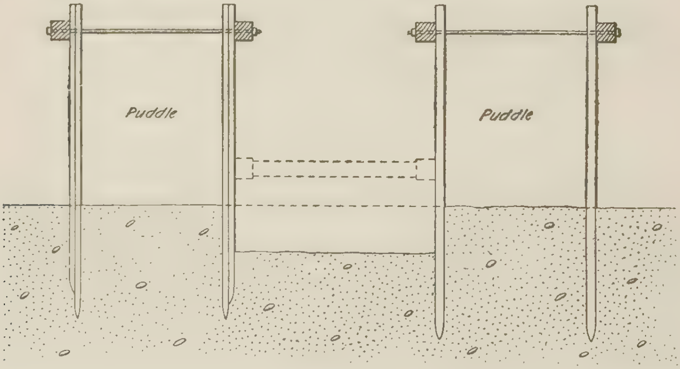

FIG. 16

the interior, and, if necessary, additional wales and cross-struts, as shown by dotted lines, may be added as the water is lowered, to resist the pressure inwards.

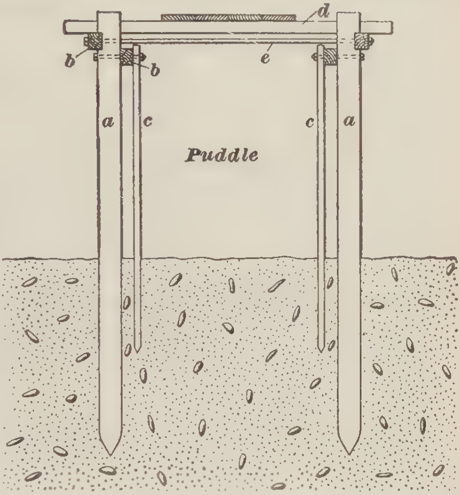

FIG. 17

45. For use in still deeper water, particularly where there is a strong current, the construction illustrated in Fig. 17 is employed. In this figure, a cross-section of one wall is shown. Two rows *a*, *a* of ordinary piles, with a suitable space between them, are first driven around the area to be enclosed. Wales *b*, *b* are then bolted to the sides of these piles, the outer lines being placed somewhat higher than the inner ones, to support cross-timbers *d* used for carrying a floor or runway. Two rows *c*, *c* of sheet piling are then driven along the inner wales, to which the tops of the sheet piling are spiked. Long bolts *e*, extending through the outer wales, prevent the spreading of the structure by the pressure of the puddle. The space between the sheet piling is then filled with puddle.

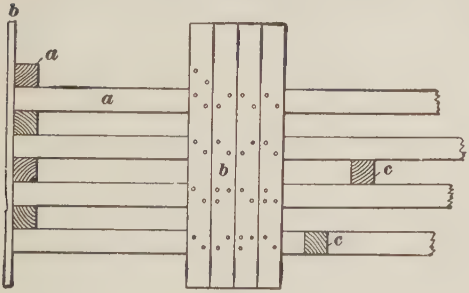

FIG. 18

46. Another typical form of coffer dam is constructed of cribwork, as illustrated in Fig. 18. It consists of a wall of timber cribwork built of square timbers *a*, *a* drift-bolted together at their intersections, and a sheeting of plank b, b spiked to the outer side of the wall. The illustration shows the details with sufficient clearness, and no further description is necessary.

Coffer dams of this kind may be built on land, launched, floated into position, and sunk to the bottom by appropriate means. If the sheeting is fairly well seasoned, with dressed edges, and the plank is put on with as close joints as possible, the swelling of the wood when the structure is placed in the water will make the joints practically water-tight. Sometimes, the timbers a, a are cut half away at the corners so that the parallel sticks are in close contact, and the joints are then calked to make them water-tight. Cross-timbers c, c may be put in occasionally to brace the sides of the coffer dam against each other.

47. Design of Coffer Dams.—The two most important requisites in a coffer dam are that it shall have sufficient strength to resist the pressure of the water on the outside when that on the inside is pumped out, and that it shall be water-tight. Since the intensity of the pressure increases with the depth, not only must the lower parts of the structure be made stronger, but greater care must be taken to prevent leakage. The total pressure P, in pounds per lineal foot, on the side of a coffer dam is found by the following formula (see *Hydrostatics*):

$$P = \frac{62.5\,h^2}{2}$$

in which h is the depth, in feet, of the submerged part of the coffer dam.

The intensity of pressure at any depth h_1 below the surface is $62.5\,h_1$ pounds per square foot, and the center of pressure is at a depth of $\frac{2}{3}\,h_1$ from the surface.

The stresses on coffer dams and their several parts are determined by the application of the mechanics of beams. Where main piles are driven, as in Fig. 17, they act as cantilever beams; where the surface material into which they are driven is soft, their capacity is decreased, and their strength should be figured with a large factor of safety.

The sides of the coffer dam may be braced against each other as shown in the preceding figures, but too many struts of this kind obstruct the laying of the masonry, and it is accordingly desirable to use as few of them as possible. As the masonry reaches them, they may be cut away and short struts resting against the masonry substituted.

48. The necessary thickness of the wall of puddle will usually determine the width of the double walls of the coffer dam. No definite rule can be given for the thickness of the puddle wall, as it depends so largely on the character and efficiency of the puddle material available. It should never be less than 4 feet, even in shallow water. An empirical rule given in Trautwine's "Pocketbook" is to make the thickness three-fourths of the height, but never less than 4 feet. This rule seems to give good results with puddle of average quality and for moderate depths. For use in water 12 feet deep, the rule would require a width of 9 feet of puddle, which is probably more than most engineers would think it necessary to use.

49. Quality of Puddle.—The effectiveness of a coffer dam in excluding water depends not only on the thickness of the wall of puddle, but also on the quality of the puddle itself. Stiff, tenacious clay, capable of being rammed into a dense mass, would seem at first thought to be the most desirable material. It is often used with success, but it has the defect that, if the water once penetrates it in a stream, however small, the current will continually enlarge the opening, the stiffness of the clay preventing it from falling into and closing the channel. Experience has shown that a mixture of gravel, sand, and clay makes the most effective puddle material, the clay being barely sufficient in quantity to fill the voids in the gravel and sand. If from any cause a rill of water should start through a mass of this material, the gravel and sand will cave into the channel and close it up effectively.

50. Water-Tightness of the Structure.—When the coffer dam is complete, pumps are put in operation to remove the water from its interior. This affords the first test of the water-tightness of the structure, and, if it is not satisfactory, the leaks must be looked for, and such expedients as the conditions suggest must be applied to stop them. It will seldom be found that a coffer dam is perfectly tight, but, if the leakage is not too large to be controlled by the pumps, the latter may be kept in operation and the water thus kept down so as to permit the construction of the foundation.

51. Several forms of interlocking steel sheet piling are now available, and are particularly valuable for coffer-dam work. This piling has a much greater strength than wood piling; the interlocking feature insures a practically water-tight joint between adjoining members; and, although its first cost is much greater than that of wood, it may often prove more economical, particularly in deep water.

52. Deep coffer dams, exposed to great pressure of water, require special care to make them water-tight. Even if the walls do not leak, the water may be forced under them or through the soil beneath them, unless the sheet piling extends to a considerable depth and the separate piles are driven in close contact with one another for their whole length. To secure the latter requisite, some form of tongued-and-grooved sheet piling is generally used, as described in Art. **27**, and it is driven, if possible, to the rock or other hard and impervious material which is to form the subfoundation. Even then the water may rise through subterranean passages or through fissures in the rock in such quantities as to be very troublesome. Such cases must be dealt with as the conditions and possibilities suggest. Sometimes, a leak may be totally or partly stopped by depositing a bank of puddle around it of sufficient width to blanket and close off any water passages in the near vicinity of the work.

53. Dredging.—If the stratum it is proposed to use for subfoundation is overlaid with soft and pervious material, the whole area to be occupied by the coffer dam is often dredged away before the coffer dam is built. Where crib coffer dams like that shown in Fig. 18 are used, they may often, after they are in place, be sunk to a considerable depth by dredging out the interior, allowing the coffer dam to sink by its own or by added weight. This method is similar to that used for sinking crib foundations, which will be described further on.

PNEUMATIC PROCESSES

THE USUAL PROCESS

54. General Description. The carrying of foundations to great depths under water or in water-bearing soils involved very serious difficulties until the development of the **pneumatic process.** By the aid of this process, foundations are now carried to depths as great as 100 feet below the surface with safety and the certainty of success. This process utilizes air pressure to facilitate the sinking of a foundation structure, called a **caisson,** through the soil, to bed rock or other hard subfoundation; the proposed structure is built up on this caisson. In the earlier pneumatic foundation work, a partial vacuum was created in the closed caisson, and the difference of pressure between the outer air and that in the caisson was utilized to force the latter into the ground. This vacuum process is now seldom used, and need not be further considered.

55. At present, compressed air is used principally to counteract the pressure of the water, and prevent the latter from flowing into and filling the caisson. The principles involved and the practical working of the process can best be explained by reference to the illustration in Fig. 19, which shows a vertical cross-section through the caisson. The caisson itself may be likened to a box without a lid, placed upside down. Its sides are formed of several thicknesses of heavy timbers tied together with screw and drift bolts, the bottom part being of triangular shape, forming a sharp edge, called the **cutting edge,** which is usually shod with iron. The cover, called the **deck** of the caisson, is formed of several, often a dozen or more, courses of timbers crossing each other at angles usually of 90°, and the whole is thoroughly tied together with bolts. In large caissons, one or more trusses or partitions crossing from side to side give the structure additional strength and stiffness. The interior open space is called the **working chamber.** The walls and roof of this chamber are made as nearly water-tight and air-tight as possible by lining them with courses of plank, the joints between which are calked with oakum and pitch. On the deck is usually, though not always, constructed a water-tight enclosure *D D*, called the **coffer dam**, the object of which is to permit the laying of the masonry below the surface of the water, if found desirable. This coffer dam may be but a few feet in height to protect the laying of the first few courses of masonry, or it may be carried to the full height of the surface of the water when the caisson has reached its permanent position. Through the deck, communicating with the working

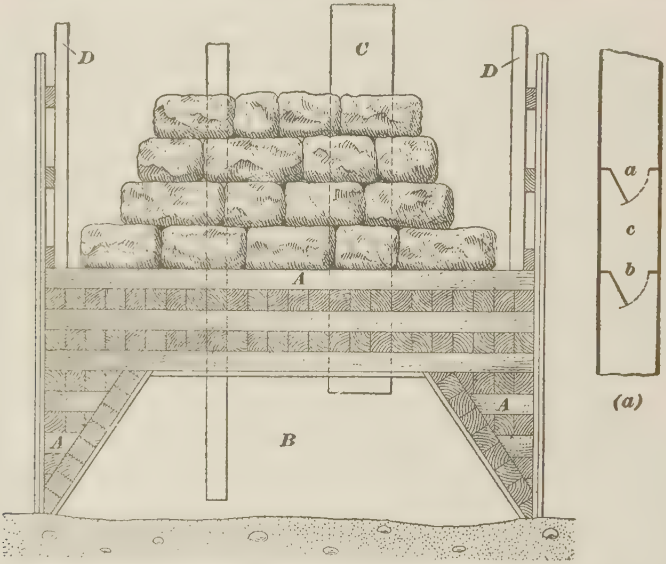

Fig. 19

chamber, are inserted a number of large tubes or shafts built of steel plates, which are carried up as the sinking of the caisson and the building of the masonry progress. The most important of these shafts is called the **air-shaft,** which is used for the entrance and exit of the men to and from the working chamber, and sometimes for the bringing out of excavated material. Two or more such air-shafts may be provided in large caissons. Situated in the air-shaft is the **air-lock,** the use of which will be explained presently. Other tubes and pipes are for the forcing out of water and semifluid excavated material, or for conveying concrete and other materials into the working chamber.

Referring to Fig. 19, *A*, *A*, *A* represents the caisson structure; *B* is the working chamber, and *C* is the air-shaft. These air-shafts are constructed with differing details, but the principle involved is the same in all.

The principle of the air lock will be understood from Fig. 19 (*a*). The lock consists of a chamber *c* separated from the upper and lower portions of the air-shaft by doors *a* and *b*, which are fitted to close air-tight. The air pressure in the working chamber and in the lower part of the air-shaft may be from 10 to 40 pounds per square inch higher than that in the upper part of the air-shaft, which is in open communication with the atmosphere. A man wishing to enter the working chamber passes down the air-shaft through the upper door of the air lock, the lower door being closed. When he is inside the air lock, the upper door is closed, the compressed air from below is slowly admitted into the lock through valves for the purpose, and when the pressure in the lock has become equal to that in the working chamber, the lower door is opened and the man continues his descent. If he wishes to return, the operation is reversed; the upper door being closed, the lower one is opened; he enters the lock and the lower door is closed; the lock is then connected through valves with the atmosphere; the compressed air in the lock escapes until the pressure is the same as that outside, when the upper door is opened and the man continues his ascent.

56. Sinking the Caisson.—Pneumatic caissons may be used either on land or in water. If for use in water, the structure may be built on land, launched when completed, floated to the site, and moored into the exact position where it is to be sunk. Mooring or guide piles are usually driven to control its sinking until it rests on the bottom. The laying of masonry on the deck is begun, and, as it progresses, the additional weight gradually sinks the whole structure until it rests on or sinks into the clay, sand, or mud of the bottom. The operation of further sinking the caisson is conducted as follows: When the walls of the caisson have penetrated the bottom material so as to seal the working chamber, air is gradually forced into it through a pipe provided for the purpose, driving out the water either under the walls of the caisson or through pipes passing upwards through the deck. The air pressure being maintained, the water is prevented from reentering the chamber, and the laborers descend through the air-shaft and begin the work of excavation. During this time, the building of the masonry has continued, the increasing weight being partly supported by the pressure of the air below and partly by the material under the edge of the walls of the caisson. As the excavation progresses, the material being removed from under the edge of the caisson sides, and the weight of the masonry on top of it increasing, the whole structure gradually sinks until the rock or other hard material is reached, the air pressure employed increasing with the depth so as to balance the water pressure from without.

57. The material to be excavated within the working chamber may be handled with picks and shovels, or by any of the other usual methods. If large boulders are encountered or if the caisson strikes solid rock on one side before the other, blasting may be employed.

Some care and skill are required to guide the caisson and keep it level. Various devices have been successfully used to move the structure horizontally even after it has been deeply buried. If from any cause it gets out of level, the material is excavated more freely from under the cutting edge of the high side, or the load on that side is increased to force it down more rapidly.

58. Removing Excavated Material.—Formerly, the excavated material was carried out through the air-shaft, but in the more recent work only rock, boulders, and such material are removed in this way. Sand and ordinary gravel are forced out through a pipe, called a **sand lift,** passing up through the deck of the caisson and through the masonry, its lower end reaching to the floor of the working chamber. The excavated material, mixed with water to make it semifluid, is deposited around the mouth of this pipe, and the compressed air forces it in a continuous stream upwards through the pipe, which carries and discharges it wherever it is to be disposed of.

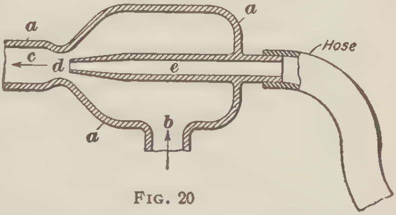

FIG. 20

59. Another very effective device for removing mud and sand is the **mud-pump,** which works on the principle of the common ejector. The current of water to operate it is supplied by powerful pumps outside the caisson. One form of mud-pump is illustrated in Fig. 20. It consists of an outer shell *a a a*, having an inlet *b* and an outlet *c*, the latter having a conical throat *d*. Entering the shell opposite this outlet is a pipe *e*, with its open end or nozzle terminating in the throat of the shell. The other end of this pipe is connected to a flexible pipe or hose, which may be moved about as required. When a stream of water under considerable pressure is forced through the pipe *e*, it escapes with high velocity through the pipe *c*, creating a partial vacuum in the shell *a a a*, by which the material is drawn in through *b*, carried forwards into the forced current of water, and discharged as in the sand lift previously described. The material fed to the mud-pump must be broken up or pulverized and mixed with sufficient water to make a semifluid mud.

60. Finishing the Foundation.—When the caisson has been carried down to the rock or other hard material on which it is designed to rest, and its cutting edge is firmly seated thereon, the working chamber is filled with concrete, or sometimes with sand, the shafts and tubes are filled and closed, and the foundation is complete.

61. General Remarks.—The caisson method of sinking foundations is now considered more satisfactory, efficient, and economical than any other for large foundations that must be carried to depths of 40 feet or more. Its application is limited, however, to depths of about 100 feet below the surface of the water, as the human system cannot endure air pressures due to greater heads. Formerly, the sickness and mortality among laborers made work in the working chamber a dangerous occupation. The **caisson disease**, as the peculiar disease caused by this work is called, was not then well understood. With proper hygienic care, it is now of infrequent occurrence, and seldom fatal.

When the working chamber reaches such a depth that the air pressure is more than two atmospheres, laborers can work only a limited length of time; and when the depth is over 90 feet, they should not be exposed to the pressure for more than 2 hours at a time, nor for more than two such periods, or "shifts," in each 24 hours.

62. The pneumatic-caisson process may be used also on land, when underground water makes any other process difficult or ineffective. The procedure is essentially the same as previously described.

The plant required for the pneumatic-caisson process is rather extensive. It embraces a complete outfit of boilers, powerful air compressors, and force pumps, together with the usual outfit for laying masonry and disposing of excavated material. In marine work, this plant is usually placed on barges moored around the work.

63. It is impossible to compute, in advance, with any degree of accuracy, the quantity or weight of masonry that will be necessary to force the caisson downwards, as the conditions are likely to vary in different cases. The forces that resist the downward movement are the air pressure in the working chamber and the friction of the sides of the caisson on the walls of earth through which it passes. The air pressure required to keep the chamber free from water is somewhat less than that required to balance the column of water above the cutting edge, because of the friction of the water in passing through the material directly under the cutting edge. The friction of the caisson on the earth surrounding it appears to vary within very wide limits, but it is usually much less than the ordinary friction between wood and clay. The presence of water between the two surfaces, and the escape of more or less air, which naturally follows up along the side of the caisson, tend to reduce greatly the usual coefficient of friction. The records show that this frictional resistance may vary from 300 to 600 pounds per square foot of the exposed surface of the caisson.

The rate of sinking also varies with varying conditions, but mainly with the speed with which the material may be excavated in the working chamber and disposed of. If the caisson can be lowered $1\frac{1}{2}$ feet per day, working 24 hours, this may be regarded as good progress.

OTHER PNEUMATIC PROCESSES

64. Pneumatic Piles.—When the expression *pneumatic foundations* is used, the system just described is usually referred to, unless otherwise stated. However, other pneumatic methods, some of them older than the one prevailing now, are sometimes used. The name **pneumatic piles** is commonly applied to hollow cylinders sunk by the pneumatic process. The cylinders may be of any desired diameter, and may be made of riveted sheet metal, or of cast-iron rings or segments with inside flanges to bolt them together. The cylinders are similar in all respects to the tubes described in Art. **35**, except that they are provided with air locks, which are placed near the bottom of the tube, as this position offers the advantage that additional sections can be added to the top of the cylinder as it sinks, without disturbing the air lock. The bottom of the cylinder is usually provided with a stiffened cutting edge, and is flared out slightly to make a space in the clay somewhat larger than the body of the cylinder, in order to reduce the friction against its sides. Other details, and the method of sinking, do not differ materially from those of the pneumatic caisson. The filling of the cylinder with masonry is not begun until the cylinder has been sunk to its final position; then sufficient concrete is placed in the bottom to seal it against water, the air lock is removed, and the remainder filled in the open air.

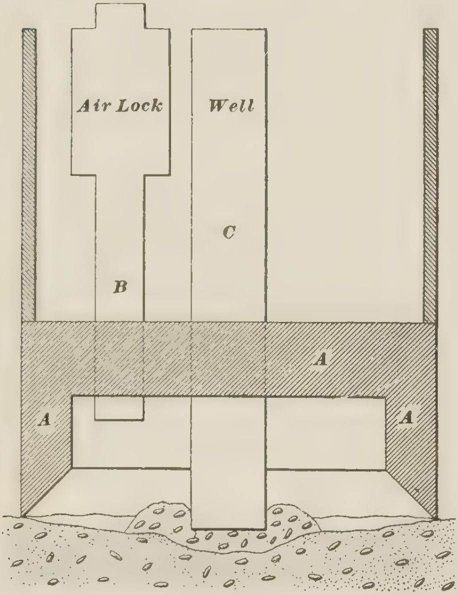

FIG. 21

65. It will be observed that these cylinders have not, as has the pneumatic caisson, the weight of the superimposed masonry to force them down, and difficulty is sometimes experienced in causing them to sink as rapidly as the material inside is excavated. Heavy weights may be placed on the top of the cylinder to assist in forcing it down, or the vacuum process may be employed to facilitate the sinking. If the friction is great and the cylinder fails to sink, a door opening upwards in the air lock may be closed and the air pumped out to form a partial vacuum in the cylinder, when the greater air pressure on the top of the air lock will force the cylinder down.

66. Water Column.—Another modification of the pneumatic-caisson process, formerly much used, relates to the method of removing the excavated material. Its principle will be understood by referring to Fig. 21, in which *A A A* is the caisson and *B* the air-shaft, with its air lock for the ingress and egress of the workmen. A well, or **water column,** *C* extends through the caisson to a depth a little below the plane of the bottom of the caisson. This well is filled with water to a sufficient height to balance the air pressure in the working chamber. The excavated material is piled around the well, into which it flows; it is then removed by some form of dredge like the common clam-shell bucket. The superior capacity and greater economy of the sand lift and the mud-pump, as now developed, for the removal of excavated material is such that the water-column method which formerly was much used, is not extensively used at present.

RIPRAP FOUNDATIONS

67. Simple Riprap Foundations.—Among the methods used to place foundations in water without freeing the site of water, riprap foundations are the most simple and primitive. **Riprap** is a name applied to a mass of broken rock deposited at random, or with little or no attempt at regular arrangement. Its use is commonly restricted to the protection of river banks or insecure foundations from erosion or undermining, but many foundations for bridge piers have been constructed with this material. Masses of rock as large as can be conveniently handled are thrown overboard from barges until a pile is accumulated reaching to, or nearly to, the surface of the water and having a top area considerably larger than the base of the pier. The surface is rudely leveled off with smaller fragments, and the masonry begun without further preparation. Settlement, sometimes irregular, will occur as the masonry progresses; this may be partially corrected by varying the thickness of a course of stone. Later, additional riprap may be deposited, enlarging the mass and building it up a few feet around the base of the pier.

68. Riprap foundations are, of course, very crude, and are not to be recommended, but it is nevertheless a fact that many railroad and highway bridges in the United States rest on such foundations, and that the percentage of failures has been small. In time, the currents of water wash gravel and soil into and fill the cavities between the masses of stone, thus forming comparatively solid and strong masses. Where rock is plentiful, these foundations are comparatively inexpensive, as they require no costly plant and only the crudest engineering skill.

69. Rock-Filled Timber Crib Foundations.—Timber cribs filled with riprap may be considered the next step in advance of simple riprap foundations. In these, a wooden house-like structure, without floor or roof, is built of round logs or squared timbers, floated into position, sunk, and filled with broken rock, the masonry resting on its top. The cribs are often built of round logs notched or flattened at the ends and intersections and secured together by drift bolts. The timber part of the structure must be permanently below the water; otherwise its top will soon decay. Sometimes the cribs are constructed with a close floor of timber or plank, but as a rule this is unnecessary.

Foundations of this kind have the advantage of cheapness where timber is plentiful and rock scarce, since, their sides being vertical, less rock is required. Besides, they offer less obstruction to navigation.

OTHER METHODS OF BUILDING FOUNDATIONS UNDER WATER

70. Open-Caisson Foundations.—The term *caisson*, as used in foundation work, is now commonly applied to any movable box-like structure having one open side and used to exclude the water from a foundation during its construction. In **open-caisson foundations**, the open side of the box is up; in pneumatic caissons, it is down. The open caisson is a water-tight box sunk in the water until its bottom rests on the bed of the body of water, the masonry being built within and directly on its bottom. This description is general; in practice, many modifications of form and many methods of construction and operation have been employed.

The caisson is usually a very large box, whose horizontal dimensions are a few feet larger than the designed base of the masonry structure, whose bottom is strong enough to resist the upward pressure, and whose sides are able to resist the lateral pressure of the water when sunk to the desired depth. The floor is composed of one or more courses of timber laid closely side by side. If two or more courses are used, the timbers in alternate courses cross each other at right angles, the joints being calked or otherwise made water-tight. The plan for the construction of the sides depends largely on the depth and the pressure of water to be provided against.

The caisson may be built on land, launched, and floated to and securely moored into its correct position, and the laying of masonry begun on its floor. The caisson is thus soon sunk to its permanent position on the bottom, and the construction of the masonry continued until its top is above the water, when the sides of the caisson, being no longer needed, may be removed.

71. Combined Crib and Grillage Foundations. Where timber is plentiful and cheap, the submerged foundation may be constructed of it wholly, and the structure may be a combination of the grillage and the crib. The bottom courses may be built of timbers packed closely together to secure a large bearing area, and the top courses of similar construction to form a close platform for the masonry. In the intermediate courses, the timbers may be spaced with reference only to their strength to resist crushing due to the weight on them; and this part of the structure may resemble a grillage with large open spaces, which may or may not be filled with broken rock. Such a foundation is simply a large timber platform resting on the bottom, with its top approaching as near as desired to extreme low-water surface. It is handled and built on in very much the same manner as described for the open caisson.

72. Preparation of the Bed or Subfoundation. The foundations described in Arts. **67, 69, 70,** and **71** have been spoken of as simply placed on the bed of the stream, which is assumed to be sufficiently level and firm for the purpose in its natural condition. This is, however, not always the case. The bottom may be uneven, or it may be composed of sand or silt, which, under the action of strong currents of water, may be undermined, allowing the foundation to be impaired or destroyed. Dredging is therefore often employed either to level the bottom or to excavate a pit into which the foundation may be sunk to such a depth below the bed of the stream that it will be, in a measure at least, secure from undermining. In some cases, crib and open-caisson foundations have been sunk to very considerable depths in sand and silt, by pumping or otherwise excavating the material from underneath the foundations through openings or shafts for the purpose, the foundation being forced downwards into the cavity by its own weight and that of the partly constructed masonry on it. In this way, crib foundations have been sunk to depths of as much as 80 feet below the bed of the stream.

73. Tubular Foundations.—Tubular or cylinder foundations, as described in Art. **35,** are frequently employed in water, the material being excavated from their interior by dredging. This method, or some modification of it, is practically the only one available where the depth that must be reached is very great, since the pneumatic process is not applicable to depths much exceeding 100 feet.

The foundations of the Hawkesbury bridge in Australia rest on a bed of compact gravel 185 feet below the surface of high water. They were sunk into place by a combination of the cylinder and open-caisson processes. The work could have been done by no other known method.

Single cylinders, in this process, are usually from 6 to 12 feet in diameter. A single one of these cylinders may be capable of carrying the required load, but because of its small diameter it will lack lateral stability. For this reason, a group of two or more are sunk near each other, and their tops are connected with steel bracing. The cylinders, when sunk to the proper depth, are filled with concrete, or sometimes with gravel, iron caps being placed on and bolted to their tops, and the structure built on these caps.

74. Combination Pile and Cylinder Process.—A form of foundation used quite successfully for bridge piers is known as the **Cushing pile foundation.** Piles are first driven as close together as practicable, the outline of the group being a circle of the desired diameter. The piles are cut off just below low water, so that they will always be submerged. A cast-iron or steel cylinder of the required diameter is then lowered over and around the group of piles, and forced into the soft material of the bottom as far as possible, usually not less than 10 feet. The water is then pumped out and the vacant spaces around and among the piles are filled with concrete. The process is then completed as described in the last article.

The piles give the foundation bearing power and lateral stiffness, while the steel cylinder, in addition to adding to the strength of the structure, protects it from external injuries. Groups of such individual structures may be employed if necessary.

Tubes used in foundations have been spoken of as *cylinders*. and they are usually of that form; but they may be square or elliptic, and have been so made.

75. Pile Foundations Under Water.—Where the material of the bed of the body of water will permit it, pile foundations may be used. A system of properly located and spaced piles are driven, the length of the piles being preferably such that, when the driving is completed, their heads will be above the surface of the water. They are all then cut off to a level plane by a circular saw mounted on a vertical shaft. A timber caisson whose bottom is a platform made of two or more courses of timbers is then floated and moored into exact position and sunk as heretofore described for open caissons, so as to rest on the tops of the piles. To prevent erosion around the piles and under the structure, riprap may be deposited around the caisson when the latter is in place.

76. Concrete Foundations Under Water.—Since hydraulic concrete sets or hardens perfectly under water, it is a most valuable and economical material for use in the construction of subaqueous foundations. With reasonable care, the freshly mixed concrete may be deposited without injury in water of great depth. If dumped loosely into the water and allowed to sink through it, the mortar will be washed out of the stone or gravel, and its usefulness will be destroyed. Several methods are employed to avoid this, the principal of which are as follows:

1. The concrete may be placed in partly filled, loosely woven cloth bags, the bags protecting the concrete while passing through the water. These bags, unopened, are placed by a diver in regular order in the foundation, being built up very much as blocks of stone in masonry. Sufficient mortar exudes through the material of the bags to cement them firmly together when in place.

2. Specially designed covered boxes or buckets having at their bottom a gate that may be opened by pulling a line, or that opens automatically when the bucket reaches the concrete already laid, are used. These buckets are handled by a derrick. When filled and closed, they are sunk through the water at the spot where the concrete is to be deposited, the gate is opened and the concrete thus deposited in quiet water in such a manner that the materials are not separated. This method is very expeditious and satisfactory and is largely employed.

3. A wood or metal tube, called a **chute**, long enough to reach from the floor of the foundation to a convenient distance above the surface of the water, is used. The concrete is filled into the top of this tube and thus conveyed to the point where it is to be deposited, the tube being moved

about as required to deposit the concrete regularly. If the tube is kept constantly full of concrete, water is excluded and the concrete may be thus put in place without injury.

FREEZING PROCESS FOR FOUNDATIONS

77. A process for sinking foundations through quicksand or semiliquid earth, called the **freezing process,** was invented by F. H. Poetsch, of Austria, and is therefore sometimes called the **Poetsch process.** It is substantially as follows:

The space to be excavated is surrounded by a series of vertical pipes sunk by the water-jet process. Each such pipe has a smaller pipe inside of it. The outer pipe is closed at its lower end, while the inner pipe is open at its lower end, and does not exactly reach to the bottom of the outer pipe. The several outer pipes are so connected that, when a refrigerating fluid is forced downwards through the inner pipe, it ascends through the space between the two pipes, is conducted thence to the next inner pipe, and is thus forced to circulate through the whole system of pipes. The refrigerating liquid is first reduced to a temperature much below the freezing point of water by apparatus similar to artificial-ice machines. The material around and between the pipes is, in time, frozen into a solid mass, which acts as the wall of a coffer dam in excluding the water or semiliquid mud from the space where the foundation is to be built. The material may then be excavated by ordinary methods. The process is necessarily slow and very expensive, but it has been successfully employed in a few cases where all other expedients had failed.

RETAINING WALLS

DEFINITIONS

1. **Retaining walls** are employed to hold the face of a bank in position, thus preventing it from sliding and caving. Specifically, a retaining wall is a wall built to hold in place a filling or backing of earth deposited behind it after the wall has been built.

2. The bottom of a retaining wall is called the **foot** of the wall. The front and back sides of the foot are called the **toe** and **heel**, respectively. In Fig. 1, *ef* is the foot of the wall *W*; *e* is the toe, and *f* the heel.

3. A **vertical wall** is a wall in which both the front and the back are vertical.

4. **Sea walls, or wharf walls,** are masonry walls built along the water front. They sustain a pressure of earth from the back and take the wash of the water at the face. They are often required to support building walls, as in the case of ferry houses and wharf sheds, in which case they are subjected to considerable pressure from heavy cargoes.

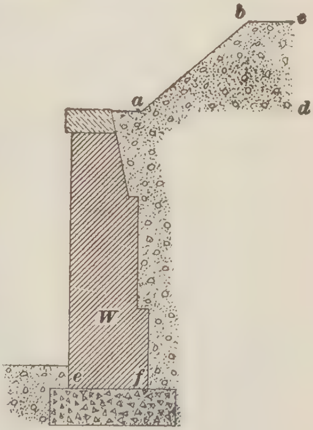

Fig. 1

5. A **face wall** is the same as a retaining wall, with the exception that the term is usually applied to a wall built

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against the vertical face of natural earth formed by an excavation. The earth, being in its natural condition, is firm and solid, and is not so likely to slide and cave as filling.

6. Buttresses are piers of masonry built at intervals on the face of a wall, as *a*, *a*, Fig. 2. They are usually thicker at the bottom than at the top, and are employed to reinforce the wall by adding to its stability, and to prevent bulging. At present, buttresses are little used.

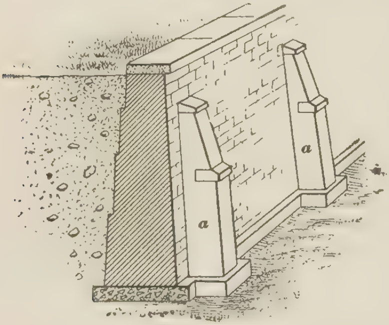


Fig. 2

7. Counterforts are projections or buttresses placed at the back of a wall, to supply additional weight in order to resist the overturning of the wall. Their utility is doubtful when built in connection with solid walls; but, when used with walls of reinforced concrete, they serve to tie a wide base to a thin-faced wall, so that the weight of the filling assists in maintaining the stability of the wall (see Fig. 3).

8. The terms **filling** and **backing** are understood to mean loose earth, gravel, or sand dumped to fill in an excavation back of a retaining wall.

9. Surcharge is that part of an earth embankment that is above the level of the top of the retaining wall. The earth shown in section at *abcd*, Fig. 1, is the surcharge.

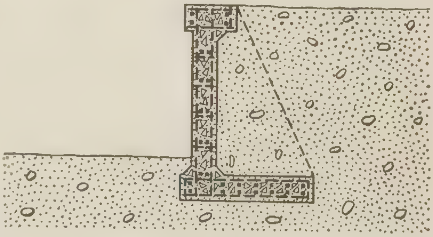


Fig. 3

10. Land ties are tension bars or wire-rope guys secured to the back of a retaining wall and carried through the earth to masonry, large stones, plates, or timbers embedded in the earth or filling, to which they are fastened. Any stone, plate, or timber used for this purpose is called a *dead man.* Land ties are never used in new work of good construction; they are a makeshift used principally to secure an old wall that shows signs of failure. Even for this purpose their usefulness is questionable.

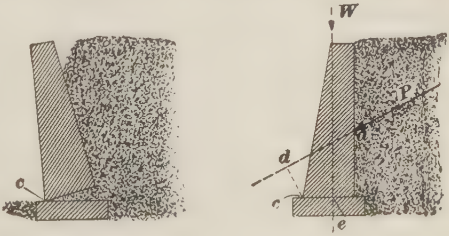
STABILITY AND DESIGN OF RETAINING WALLS

CAUSES OF FAILURE—COEFFICIENT OF FRICTION

11. Aside from the possible failure due to the settlement or upheaval of the foundation from softness of the supporting soil, or from frost or other cause (and this should be considered as failure of the foundation rather than of the wall itself), retaining walls may fail in three ways; namely, by *overturning*, by *bulging* in a vertical plane, or by *sliding* laterally.

12. Moment of Stability.—The **moment of stability** of a retaining wall is the moment of the weight of the wall about its toe, and is obtained by multiplying the weight of the wall by the distance from the toe to a vertical line passing through the center of gravity of the wall.

13. Overturning.—The overturning of a retaining wall—the most likely cause of failure—occurs whenever the moment of the pressure of the earth about the toe is greater than the moment of stability of the wall. Fig. 4 (*a*) illustrates the tendency of the wall to turn about its toe *c*. Referring to Fig. 4 (*b*), the pressure of the earth is represented by *P*, its direction being shown by the arrow, and its point of



(*a*) FIG. 4 (*b*)

application, or the location at which it acts against the wall, being denoted by a. The force P tends to turn the wall about the toe c. In order that this tendency may be counterbalanced by the weight W of the wall, the moment of P about the toe c must not exceed the moment of W about the same point; that is, $P \times cd$ must not be greater than $W \times ce$, and the greatest value that P can have, consistent with the stability of the wall, is that obtained from the equation $P \times cd = W \times ce$.

14. Bulging.—The bulging of a retaining wall, as shown in Fig. 5, will occur when the top and bottom of the wall are held in place, as, for instance, in a cellar wall or in the wall of an area supported at the top by arches abutting against the wall, as shown at a. Bulging is likely to occur when the wall is thin, the point of greatest bulge usually being at about one-third the height from the footing, as at b; it is also sometimes caused by dumping the filling against a *green wall*—that is, a wall in which the cement or mortar has not had time to set properly.

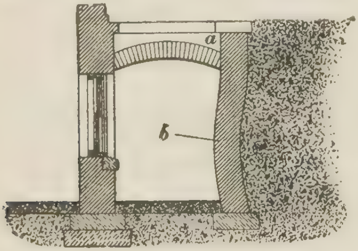


Fig. 5

A slight bulging in a retaining wall is not a serious matter, and need not be considered dangerous unless it exceeds $\frac{1}{4}$ inch for each foot in thickness of the wall at the point where the greatest bulging occurs.

15. Sliding occurs where the wall is sufficiently heavy and wide at the base to prevent overturning, but where, owing to the nature of the soil or the design of the footings, the friction between the base and either the footing or the soil is not sufficient to prevent a lateral movement of the wall. A wall is liable to fail by sliding in any horizontal joint, but, as the mortar has considerable adhesive value, any lateral movement that may occur is more liable to be between the masonry and the soil.

16. Coefficient of Friction and Angle of Repose. The subject of sliding friction is fully treated in *Analytic Statics*, Part 2, and here it will be sufficient to give the values of the coefficient of friction f and angle of friction or of repose Z, for the materials most commonly used in the construction of retaining walls. As explained in *Analytic Statics*, $f = \tan Z$.

Table I gives the coefficients of friction and angles of repose between different classes of masonry and between the soil and masonry for different conditions.

TABLE I

COEFFICIENTS OF FRICTION AND ANGLES OF REPOSE

Material	Coefficient of Friction	Angle of Repose Degrees
Fine-cut granite, on same, dry	.60	31
Fine-cut granite, on rough-pointed granite, dry	.65	33
Rough-pointed granite, on same, dry	.70	35
Well-dressed soft limestone, on same, dry	.75	37
Concrete blocks, on same, dry	.65	33
Concrete blocks, on fine-cut granite, dry	.60	31
Common brick, on same, dry	.65	33
Common brick, on well-dressed soft limestone, dry	.65	33
Common brick, on well-dressed hard limestone, dry	.60	31
Common brick, on same, with slightly damp mortar	.75	37
Hard brick, on same, with slightly damp mortar	.70	35
Hard limestone, on same, with slightly damp mortar	.65	33
Common brick, on same, with fresh mortar	.50	27
Well-dressed granite, on same, with fresh mortar	.50	27
Granite, roughly worked, on dry sand and gravel	.50 to .60	27 to 31
Granite, roughly worked, on wet sand	.35 to .45	19 to 24
Granite, roughly worked, on dry clay	.50	27
Granite, roughly worked, on moist clay	.35	19

The coefficients of friction of other classes of masonry on various soils may be taken the same as those given for granite in the table.

The coefficient of friction can hardly be given more exactly than within .05; slight differences in the conditions of the materials or surfaces, which may not be perceptible to the ordinary observer, may make a very noticeable difference in the coefficient of friction. With mortar joints in masonry, the real coefficient is probably that between each of the courses of the stone and the mortar of the joint, with only a slight friction between the stones where they may be brought lightly in contact with one another at isolated points. The effect of moisture on the coefficient of friction is very well illustrated by a comparison of the coefficients for two surfaces of common brick in contact, as given in Table I: if dry, the coefficient is .65; if with slightly damp mortar, it is .75; while with fresh wet mortar, it drops to .5. As a rule, a slight dampness increases the friction, while saturation decreases the friction very materially. Clay is an exception to this rule: in it the friction is reduced by even a small amount of moisture.

TABLE II

COEFFICIENTS OF FRICTION, ANGLES OF REPOSE, AND WEIGHTS OF EARTHS

Material	Coefficient of Friction f	Angle of Repose Z Degrees	Weight Pounds per Cubic Foot
Mixed earth, dry .	.70	35	95
Mixed earth, damp	.80	39	115
Mixed earth, wet .	.40	22	115
Sand, dry	.65	33	110
Sand, wet	.05	3	125
Loam, dry	.70	35	75 to 100
Loam, wet	.50	27	90 to 120
Clay, dry	1.00	45	100
Clay, wet	.30	17	125

17. The angle of repose of loose materials corresponds to the angle of repose of the individual particles of which those materials are composed. Table II gives the coefficients of friction f and angles of repose Z of various kinds of earth, with the weight per cubic foot. These quantities refer to the friction of each material on itself—that is, to the friction of the particles on one another. These elements are important factors in determining the stability of retaining walls. For ordinary calculations, the slope of repose is taken as $1\frac{1}{2}$ horizontal to 1 vertical, giving a coefficient of friction of .67, or an angle of 34°, with which a weight of backing of 100 pounds per cubic foot is used.

18. Where the footing of a retaining wall has a level bed, as in Fig. 6 (*a*), the resistance to sliding is equal to the weight of the wall (with any superincumbent load or force) multiplied by the coefficient of friction. In order to provide greater resistance to sliding than is obtained by the frictional resistance between the masonry and the soil or base with a level bed, the bottom surface of the footing is sometimes inclined inwards, as shown by *dc*, Fig. 6 (*b*). In this case, the resistance to sliding is equal to the component of the weight normal to the base *dc* multiplied by the coefficient of friction, plus the component of the weight parallel to the base.

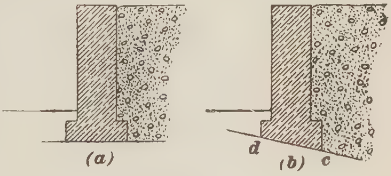


FIG. 6

If a wall is built with mortar joints, the backing should not be placed until the mortar has fully set, or there will be a much smaller resistance to sliding than may be expected after the mortar has set.

EXAMPLE 1.—A brick wall weighing 2,000 pounds rests on a level base of well-dressed soft limestone. What horizontal force will be necessary to slide the wall on the foundation, if the joint is without mortar and dry?

SOLUTION.—From Table I, the coefficient of friction for common brick on well-dressed soft limestone, dry, is found to be .65; multiplying this by the pressure on the bearing, 2,000 lb., gives 1,300 lb. as the horizontal pressure required to cause sliding. Ans.

EXAMPLE 2.—Assuming that the resultant horizontal thrust from the earth back of a granite retaining wall weighing 5,000 pounds per running foot and resting on a clay soil is 1,800 pounds in the same length of wall, will the wall be secure against sliding, and what factor of safety will it possess when the clay is dry? How will the dampening of the clay modify the safety?

SOLUTION.—For a wall of this weight to be just on the point of sliding with this pressure at the back will require a coefficient of friction of 1,800 ÷ 5,000 = .36. From Table I, the coefficient of granite on dry clay is found to be .5, and on moist clay .35. The factor of safety against sliding, is therefore, .5 ÷ .36 = 1.4, while the clay is dry. As .36 is greater than .35, the wall may slide if the clay becomes moist. Ans.

THEORY OF STABILITY

ASSUMPTIONS

19. Many theories relating to the stability of retaining walls have been evolved by eminent physicists. Although interesting, these theories have been proved by practice and experience to be unreliable. Nearly all of them are founded on three assumptions; namely, (1) that the line of rupture of the material composing the backing is a plane; (2) that the point of application of the resultant pressure on the back of the wall is the same as for water pressure; (3) that the direction of the thrust may be determined.

The first of these assumptions is known, by observation, to be false; while the others, though they seem reasonable, have not been conclusively proved.

20. In the **first assumption**, the earth at the back of the wall is considered as tending to slide along an oblique plane *a b*, Fig. 7 (*a*). There is but one character of soil—clean dry sand—that will realize this theoretical condition;

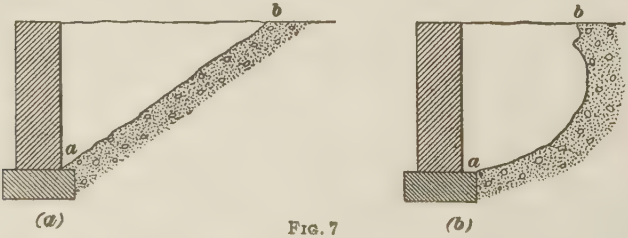


FIG. 7

all other soils possess considerable tenacity, and their line of rupture is a more or less irregular curve, such as *a b*, Fig. 7 (*b*).

The angle that the assumed plane of rupture makes with the horizontal, or the angle of repose, varies greatly with the different soils, and also with soils of the same nature containing different amounts of moisture. Damp sand or earth will stand when the slope has an inclination of 1 vertical to 1 horizontal. Roots also, serving as binders, have a large effect on the cohesiveness of the soil. The best practice in estimating the pressure of the filling on the back of a retaining wall is to consider the angle of repose for all earths and soils as 34°—that is, approximately 1½ of horizontal run to 1 of rise. This is the natural slope of dry earth without roots, and as the pressure against a retaining wall is greater with dry material than with wet, provided that the latter does not possess such a degree of fluidity as to produce hydrostatic pressure, this angle is usually assumed.

21. The **second assumption**—that the point of application of the resultant pressure is the same as for water pressure—is equivalent to considering the earth backing as a fluid, devoid of tenacity and friction. Since the nature of earth varies so materially from that of water, this assumption is far from correct, though it is probably as nearly correct as any that can be made. Whatever error this assumption contains is on the side of safety.

22. The **third assumption**, regarding the direction of the thrust of the earth, varies with the several theories regarding the stability of retaining walls. In the theory advanced by Coulomb, the direction of the earth's pressure is assumed to be perpendicular to the face of the wall; this assumption disregards the friction of the earth against the face of the wall, and therefore cannot be correct. The physicist Weyrauch determines the angle that the line of pressure makes with the wall by a complicated trigonometric formula deduced from an elaborate theory that has little practical value; for, in opposition to all practical results, it gives a greater pressure when the wall is inclined backwards toward the natural slope than when it is straight or leans forwards; it also neglects the vertical component of the earth's pressure, which certainly exists.

COULOMB'S THEORY

23. General Statement of the Theory.—The first theory (dating from the 18th century) for determining the stability of retaining walls, and the one on which most of the other theories are based, is known as **Coulomb's theory.** This is principally interesting as a study, and usually gives results that err on the side of safety, for walls that it demonstrates to be about to fall seem to be perfectly stable, and apparently possess a factor of safety of about 2. Coulomb assumed: (1) that the earth of an embankment held in place by a retaining wall tends to slide along a straight line or plane; (2) that the center of pressure of the earth acts at a point one-third of the height of the wall above the base; and (3) that the line of pressure acts perpendicularly to the back of the wall. From these assumptions, he determined, by reasoning and calculations, that the maximum pressure on a vertical wall is not due to the weight of the prism *c a b*, Fig. 8, but is caused by the wedge-shaped prism of earth *c a d* included between the vertical back of the wall and the line *a d* that bisects the angle *c a b*. The line *a b* forms an angle with the horizontal equal to the angle of repose of the material. The line *a d* is called the **slope of maximum pressure,** and the prism whose cross-section is represented by the triangle *c a d* is called the **prism of maximum pressure.**

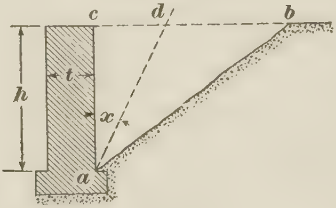


FIG. 8

24. Formulas for Vertical Walls.—

Let P = total pressure, normal to the back of a retaining wall, per unit of length of wall;
W = total weight of triangular section of earth $c\,a\,d$, Fig. 8, between back of wall and slope of maximum pressure, for same length of wall;
W_1 = total weight of wall for same length;
w = weight of backing per unit of volume;
w_1 = weight of wall per unit of volume;
h = height $a\,c$ of wall;
t = thickness of wall, in the same units as h;
x = angle between back of wall and slope of maximum pressure, or angle $c\,a\,d$.

The volume of the prism $c\,a\,d$, whose altitude or depth is **1**, is

$$\frac{a\,c \times c\,d}{2} \times 1 = \frac{h \times c\,d}{2} = \frac{h \times h \tan x}{2} = \frac{h^2 \tan x}{2}$$

Therefore,

$$W = \frac{w\,h^2 \tan x}{2} \qquad (1)$$

If the weight W is resolved into two components, one horizontal and one parallel to $a\,d$, the former is assumed to be the pressure P on the back of the wall. Therefore,

$$P = W \tan x$$

or, replacing the value of W from formula **1**,

$$P = \frac{w\,h^2 \tan^2 x}{2} \qquad (2)$$

Coulomb assumed that this pressure is equivalent to a single force of the same magnitude concentrated at a point one-third the height from the base ($= \frac{1}{3}h$). Considering this as correct, and the value of P being as just determined, the thickness required for a vertical wall that it may be stable both as to overturning and sliding can be easily ascertained.

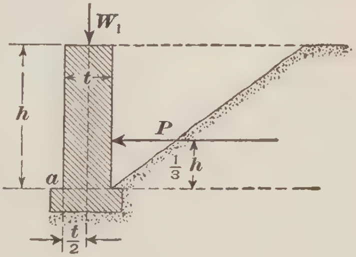


Fig. 9

25. Referring to Fig. 9, it being understood that a unit length of the wall is considered.

the moment of stability of the wall is $W_1 \times \frac{t}{2}$. The moment of P about a is $\frac{Ph}{3}$. Therefore, assuming P to be the maximum pressure consistent with stability (see Art. **13**),

$$W_1 \times \frac{t}{2} = \frac{Ph}{3}$$

But $W_1 = w_1 h t$; therefore,

$$w_1 h t \times \frac{t}{2} = \frac{Ph}{3};$$

whence $$t = \sqrt{\frac{2P}{3w_1}} \qquad (1)$$

If the value of P, as given by formula **2**, Art. **24**, is substituted, the formula just given becomes, after reducing,

$$t = h \tan x \sqrt{\frac{w}{3w_1}} \qquad (2)$$

Since cba, Fig. 8, is the angle of repose Z, we have

$$cab = 90° - Z$$

and, therefore,

$$x = \tfrac{1}{2}(90° - Z) = 45° - \tfrac{1}{2}Z \qquad (3)$$

For a slope of $1\frac{1}{2}$ horizontal to 1 vertical, $\tan Z = \frac{2}{3}$; $Z = 33°\ 42'$; $x = 45° - 16°\ 51' = 28°\ 9'$; $\tan x = .535$; and, therefore,

$$t = .535\, h \sqrt{\frac{w}{3w_1}} \qquad (4)$$

26. The limit of stability as to sliding on the base occurs when

$$P = f W_1 = f w_1 h t, \qquad (1)$$

f representing the coefficient of friction between the base of the wall and its foundation or subfoundation. Substituting in (1) the value of P from formula **2**, Art. **24**,

$$\frac{w h^2 \tan^2 x}{2} = f w_1 h t;$$

whence $$t = \frac{w h \tan^2 x}{2 f w_1} \qquad (1)$$

for the limit of stability as to sliding.

For a natural slope of $1\frac{1}{2}$ horizontal to 1 vertical, this reduces to

$$t = \frac{.14\, w\, h}{f\, w_1} \qquad (2)$$

The preceding formulas may be applied to any joint of the wall, by substituting for h the height of wall above the joint, and for f the coefficient of friction of the joints considered.

27. If the foundation is carried below the surface of the ground at the foot of the wall, the pressure of the earth in front will assist in opposing the tendency of the wall to slide on the foundation, and will also be of assistance in resisting overturning; but, unless the depth is greater than it is usually made, the increased resistance to overturning is unimportant. As the coefficient of friction for the masonry on the soil is frequently much less than that for two surfaces of masonry on each other, the assistance of the fill in front is often valuable in resisting sliding.

28. Wall With Battered Back.—For a wall with a battered back, following the assumptions made by Coulomb, the formulas should be modified by: (1) the increased overturning pressure due to the weight of the extra amount of backing $b\,c\,t$, Fig. 10, lying between the back of the wall and the vertical plane passing through its heel; (2) the decreased leverage of the overturning thrust, due to the fact that the line of action of the pressure, being perpendicular to the back of the wall, has a smaller arm ($q\,a$) than when the back is vertical ($p\,a$); (3) the reduction of the resisting weight of the wall in proportion to the thickness at the base ($b\,c\,t$ cut off from the wall); and (4) the decreased leverage, in proportion to the base, of the resisting weight of the wall due to its center of gravity k being thrown forwards ($a\,r$ is less than one-half the base).

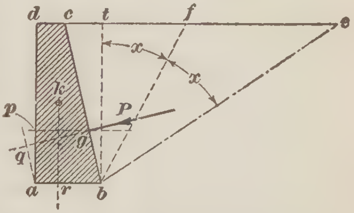


FIG. 10

All but the second of these modifications tend to increase, while this one decreases, the required thickness of the wall at its foot. The *average* thickness of wall required will be very nearly the same for a wall with a battered back as for a vertical wall.

29. Wall With Battered Face.—On the assumption of Coulomb's theory, battering the face of a wall modifies the formulas for overturning: (1) by reducing the weight of the wall in proportion to its base by cutting off $a_1 a d$, Fig. 11; and

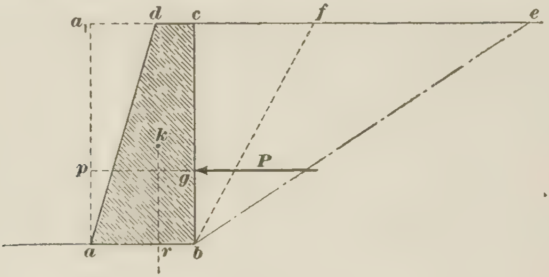


FIG. 11

(2) by increasing the leverage of this resistance to overturning ($a r$ is more than one-half the base). The net result is a small increase of base required, but a decided decrease of average thickness as compared with a vertical wall.

30. Remarks.—Coulomb's theory is unsatisfactory, because of the incorrect assumptions; because of the neglect of the friction on the back of the wall and along the plane of slope of maximum pressure, and because of the vertical pressure of the earth on the back of the wall when the latter is built with a slope or with steps. The theory is further at fault for a surcharged wall, in that in this case the slope of maximum pressure will no longer be midway between the vertical and the slope of repose. Though this theory is very imperfect, it has been given here because nearly all the theories that have been advanced for retaining walls are based on it; and, as the best theories are simply amplifications of Coulomb's, they can perhaps be better explained and understood by studying the latter first. On account of the fact that the theory, as originally proposed, is no longer used, it is scarcely worth while to give here the modifications that must be introduced in the formulas derived for vertical walls to make them applicable to battered walls. For the same reason, the treatment of surcharged walls by Coulomb's theory is omitted entirely.

Example.—A vertical retaining wall of rubble masonry, laid in cement mortar and weighing 150 pounds per cubic foot, is to be built with the backing level with the top of the wall, and 16 feet above the top of the foundation, which is at the level of the ground below. What thickness of wall will be required, by Coulomb's theory, if the backing weighs 100 pounds per cubic foot and has a slope of repose of $1\frac{1}{2}$ horizontal to 1 vertical, and the coefficient of friction of the masonry on the foundation is .65?

Solution.—To apply formula **4**, Art. **25**, we have $h = 16$, $w = 100$, and $w_1 = 150$. Therefore,

$$t = .535 \times 16\sqrt{\frac{100}{3 \times 150}} = 4.04 \text{ ft.}$$

Substituting the same values, and also that of f in formula **2**, Art. **26**,

$$t = \frac{.14 \times 100 \times 16}{.65 \times 150} = 2.3 \text{ ft.}$$

As this thickness is less than that required for stability against overturning, 4.04 feet is the proper thickness to use.

MOSELEY'S THEORY

31. General Statement of the Theory.—Moseley's theory of retaining walls is a modification of, and a great improvement on, Coulomb's theory. It takes account of the friction of the earth on the back of the wall, and also, if the back of the wall is battered, of the vertical pressure on the horizontal projection of the batter. When the back of the wall is battered or stepped, the effect of the vertical pressure of the earth resting on the slope or the steps far exceeds the overturning pressure due to the extra amount of backing.

The effect of friction is a very important factor, as it about doubles the stability of a wall under ordinary conditions, as compared with what it would be if there were no friction.

32. In Moseley's theory, the *horizontal component* X, Fig. 12, of the pressure of the filling on the wall is assumed to be as in Coulomb's theory; that is (formula **2**, Art. **24**, and formula **3**, Art. **25**),

$$X = \frac{w\,h^2 \tan^2 (45° - \tfrac{1}{2} Z)}{2}$$

The *total pressure* P is the resultant of the horizontal pressure X and the friction Y between the wall and the filling. This friction is equal to $f_1 X$, where f_1 is the coefficient of friction between the material of the wall and that of the filling; and the angle $p\,e\,x$ is equal to the angle of friction Z_1, whose tangent is f_1 (see *Analytic Statics*, Part 2). The point of application of P is assumed, as in Coulomb's theory, to be such that $b\,e = \frac{1}{3}\,b\,c = \frac{1}{3}\,h$.

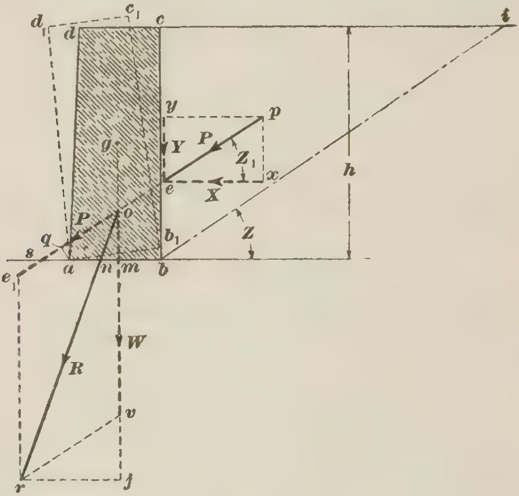


FIG. 12

The effect of friction will be better understood by considering the condition of critical equilibrium. When the wall is in that condition, the slightest increase of pressure at the back will cause overturning. In overturning about the toe a, the back of the wall rises and the wall moves forwards, as indicated by the dotted lines $a\,b_1\,c_1\,d_1$, the motion being first vertical, and gradually changing to a forward motion as the wall revolves about the toe. As the wall starts to rise, the backing starts to fall and fill the gap caused by this motion, and the pressure of the earth on the back of the wall produces a friction, represented by $y e$, whose magnitude is proportional to the normal pressure X and to the coefficient of friction f_1. Even though there is actually no motion, the power of resistance is still present to prevent overturning; it modifies the effect of the pressure of the fill on the back of the wall, and increases the stability of the wall.

The coefficient of friction f_1 of the earth against the masonry is generally taken as .65 for dry earth; this corresponds to a slope of nearly $1\frac{1}{2}$ to 1.

33. Pressure on Base of Wall.—The total pressure R, Fig. 12, acting on the base of the wall is the resultant of the pressure P and the weight W of the wall. Its magnitude and line of action are determined by the parallelogram $o e_1 r v$, in which $o e_1 = P$ and $o v = W$, the point o being the intersection of the line of action of P with a vertical through the center of gravity g of the wall. To find R by calculation, the sides $o e_1$ $(= P)$ and $e_1 r$ $(= W)$ of the triangle $r e_1 o$ are known; also, angle $o e_1 r = 180° - e_1 o v = 180° - e p x = 180° - (90° - p e x) = 90° + p e x = 90° + Z_1$.

34. If both the wall and the foundation were absolutely incompressible and incapable of fracture or crushing, the wall would be safe from overturning if the point n where the line of action of R meets the base came anywhere inside the base of the wall; and, theoretically, the pressure P could be increased until n coincided with a—that is, until the line of action of the resultant pressure R passed through the toe a. But, as explained in *Foundations*, Part 1, practical considerations require that, under ordinary conditions, the point n should fall within the middle third of the base of the wall. The theory of pressure on foundations is fully treated in that Section, which should be referred to for further particulars. It must be stated, however, that the distance $a n$ may safely be reduced somewhat from one-third to even one-fifth the width of the base, if the foundation is perfectly rigid and the masonry of the best. This will give a maximum intensity of pressure on the foundation at a $3\frac{1}{3}$ times the intensity there would be if the center of pressure were at the center of the base.

35. Stability Against Sliding.—The total pressure R on the base may be resolved into a vertical component oj $(= W + Y)$ and a horizontal thrust jr $(= X)$ tending to produce sliding on the base. This thrust must not exceed the product of the normal pressure oj and the coefficient of friction between the wall and its foundation; otherwise expressed, the angle roj must not exceed the angle of friction between the wall and its foundation, unless some external means, such as earth placed in front of the wall at the base, is employed to strengthen the wall against sliding.

Ordinarily, the friction of the back filling is disregarded in determining the resistance to sliding. It is, however, advisable to take it into account, for though latent when there is no motion of the wall, the instant that the wall begins to move, or is about to do so, whether by overturning or by sliding, the filling begins to slide—or is ready to do so—down the back of the wall, and brings the friction into action. The neglect of this factor of stability against sliding is the more readily yielded to because usually the thickness of wall required for stability against overturning gives ample weight to resist sliding, and the added help of the filling in front of the foundation, required on account of frost and other surface influences, is generally sufficient to make up for the neglected friction of the filling.

EXAMPLE.—Taking the example given in Art. **30**, with a wall 4 feet thick, and assuming the wall and foundation to be perfectly rigid and the coefficient of friction of the masonry on the foundation and on the backing = .65: (a) what are the factors of safety against overturning and sliding? (b) where is the point of application of the resultant pressure R, Fig. 12?

SOLUTION.—(a) In this case (see Art. **30**), tan x = .535; h = 16, and w = 100; therefore, referring to Fig. 12 (see formula **2**, Art. **24**),

$$X = \frac{100 \times 16^2 \times .535^2}{2} = 3{,}660 \text{ lb., nearly.}$$

Since $f_1 = .65$, we have

$$Y = .65\,X = .65 \times 3{,}660 = 2{,}380 \text{ lb., nearly}$$

Also, $\tan Z_1 = .65$, $Z_1 = 33°\,1'$, and

$$P = \frac{X}{\cos Z_1} = \frac{3{,}660}{\cos 33°\,1'} = 4{,}365 \text{ lb.}$$

To find the lever arm $a\,q$, Fig. 12, produce $p\,e$ to its intersection s with $b\,a$ produced. Then,

$$b\,s = b\,e \cot e\,s\,b = b\,e \cot p\,e\,x = \tfrac{1}{3}\,b\,c \cot Z_1 = \frac{b\,c}{3 \tan Z_1} = \frac{16}{3 \times .65} = 8.2 \text{ ft.}$$

Also $\quad a\,s = b\,s - b\,a = 8.2 - 4 = 4.2$ ft.;

$$a\,q = a\,s \sin e\,s\,b = a\,s \sin Z_1 = 4.2 \sin 33°\,1' = 2.29 \text{ ft.}$$

The moment of P about a is, therefore,

$$4{,}365 \times 2.29 = 9{,}996 \text{ ft.-lb.}$$

The weight of the wall, per foot of length, is $16 \times 4 \times 150 = 9{,}600$ lb., and the moment of stability of the wall is

$$9{,}600 \times m\,a = 9{,}600 \times 2 = 19{,}200 \text{ ft.-lb.}$$

Dividing this by the moment of P, the factor of safety against overturning is

$$19{,}200 \div 9{,}996 = 1.92, \text{ nearly. Ans.}$$

The force tending to cause sliding is the horizontal thrust X, or 3,660 lb. The frictional resistance that the wall can offer is

$$o\,j \times f_1 = (W + Y) \times f_1 = (9{,}600 + 2{,}380) \times .65 = 7{,}790 \text{ lb.}$$

The factor of safety against sliding, is, therefore,

$$7{,}790 \div 3{,}660 = 2.13, \text{ nearly. Ans.}$$

(*b*) In the triangle $o\,m\,n$,

$$o\,m = m\,s \tan o\,s\,m = (b\,s - b\,m) \tan Z_1 = (8.2 - 2) \times .65 = 4.03 \text{ ft.}$$

In the triangle $o\,r\,j$,

$$\tan r\,o\,j = \frac{r\,j}{o\,j} = \frac{X}{W + Y}$$

In the triangle $o\,m\,n$,

$$m\,n = o\,m \tan r\,o\,j = 4.03 \times \frac{X}{W + Y} = 4.03 \times \frac{3{,}660}{9{,}600 + 2{,}380} = 1.23 \text{ ft.}$$

Ans.

36. Remarks.—In actual practice, it is customary to find the magnitude and direction of the maximum pressure graphically, instead of by calculation. As the weight of the backing, and the angle of friction, as well as the strength, cohesion, and compressibility of the materials, are so imperfectly known, the results may be determined with sufficient accuracy by graphic instead of analytic methods.

37. It is to be noted that, in the foregoing example, the wall is a trifle thinner than would be required by Coulomb's theory with no factor of safety; yet, by Moseley's theory, the wall has a factor of safety of nearly 2. The difference is due to the introduction of the friction on the back of the wall.

38. Battered Back.—To ascertain the resultant of the pressure of the bank and the friction it develops when the back of the wall is battered, the angle of friction should be laid off from a line normal to the back, as shown in Fig. 13. This will make allowance for the greater overturning pressure of the backing (due to the larger wedge of earth between the back of the wall and the slope of maximum pressure, and also to the greater wedging angle between these two planes) and for the extra resistance to overturning due to the weight of earth acting vertically on the horizontal projection of the batter of the back. The net result of this deflection of the line of thrust is to reduce the overturning moment to be resisted by the weight of the wall,

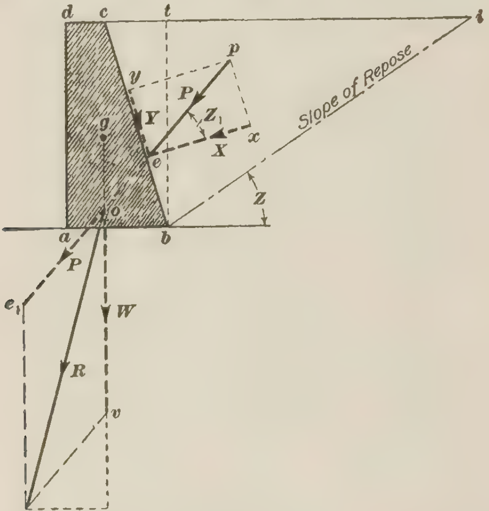


Fig. 13

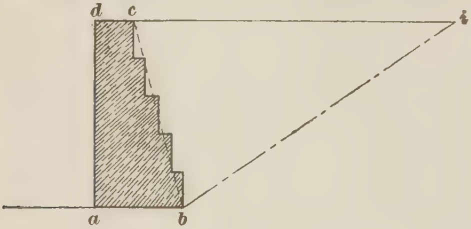


Fig. 14

so that, with the same average thickness of wall, and the face remaining plumb, the stability will be increased. If the batter of the back is sufficient, the line of action of P will cut the base, and there will be no overturning moment. The batter of the back also reduces the weight and leverage of the wall.

If the back of the wall is stepped, or offset, as in Fig. 14, the line of thrust will be the same as though the back were battered to the plane bc.

39. Surcharged Walls.—With a surcharged wall, as in Fig. 15, the weight of all the earth cbk_1 lying between the slope of maximum pressure, the back of the wall and the top of the surcharged slope should be employed instead of the smaller quantity cbk that would be used if the filling were leveled off even with the top of the wall. According to this theory, if the surcharge is carried higher than the point k_1, where the slope of maximum pressure intersects the slope of the top of the surcharge, as to u, the pressure of the filling on the back of wall is not increased thereby (see Art. 48); but, if the top of surcharge is lower than k_1, as at u_1, the weight of earth in the prism $lbcu_1$ should be used.

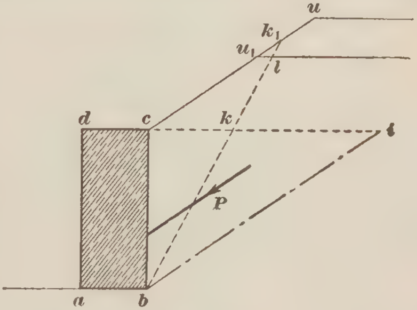


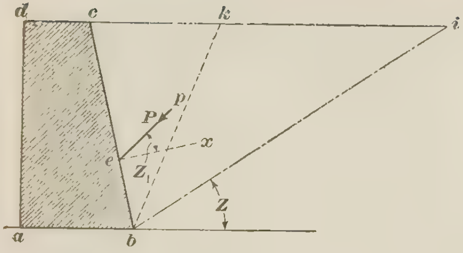
FIG. 15

If the surcharge is allowed to run over on the top of the wall, as in Fig. 16 (e), (f), and (g), only the part of the surcharge lying behind a vertical plane cz carried up from the back edge of the top of the wall should be used in determining the pressure of the filling; while the weight of the earth lying in front of this plane should be added to the weight of the wall, to find the resistance offered to overturning. In all cases, *the weight of earth to be used should be the weight of the earth lying between the slope of maximum pressure* bk, *Figs. 15 and 16, the upper surface of the soil, the back of the wall, and a vertical plane c z, Fig. 16, extended upwards from the upper edge of the back.* This general rule applies whether the earth is leveled off with the top of the wall, as *c k i*, Fig. 16 (*a*), or at a lower level, as *y k i*, Fig. 16 (*b*); whether the wall is surcharged with the toe of the slope lying at the back of the top of the wall, as *c k b* in (*c*) and *c u k b* in (*d*); surcharged with the slope running over the top of the wall, as *c z k b* in (*e*), *c z u k b* in (*f*), and *c z k b* in (*g*); surcharged with the toe of the slope back from, or below, the top of the wall, as *c y k b* in (*h*) or *y k b* in (*i*); or whether the top of the earth is lower behind the wall than its top, as *c k b* in (*j*), *y k b* in (*k*), and *c y k b* in (*l*).

40. The direction of the pressure is unchanged by the position of the top of the backing, being governed by the batter (or equivalent offsetting) of the back of the wall and by the angle of friction between the backing and the masonry.

41. The point of application of the resultant pressure is the point where a line parallel to the slope of maximum pressure, and passing through the center of gravity of the mass of earth causing this maximum pressure, cuts the back of the wall. If the cross-section of this mass of earth is triangular, with its apex at the top of the back of the wall—whether from this point the surface of the backing is level, as at Fig. 16 (*a*), or rising, as at (*c*), or falling, as at (*j*)—this point of application is at one-third the height of the wall, for the center of gravity of every triangle is at one-third its altitude measured from any side as a base; and a line passing through this point and parallel to the assumed base will cut each of the other sides at one-third its length from this base.

If the cross-section is triangular, with its apex at the back of the wall below the top, as at (*b*), (*i*), or (*k*), the point of application will, of course, be lowered to one-third the height of the wall to this apex. If the cross-section has more than three sides—whether this is caused by the top of surcharge lying below the intersection of the slope of


d
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P
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Z
a
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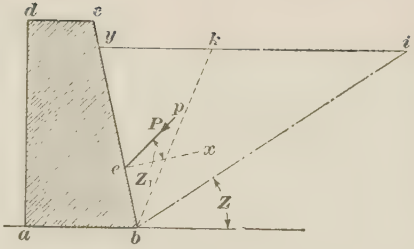
(a)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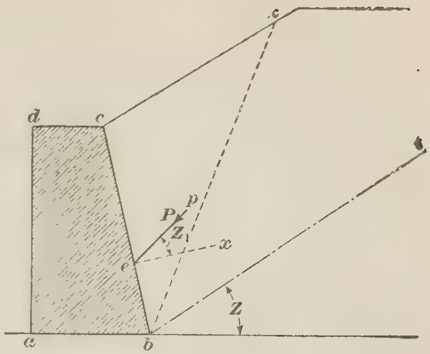
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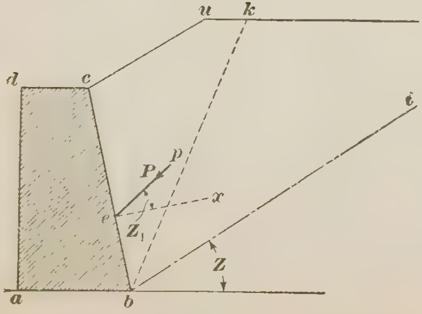
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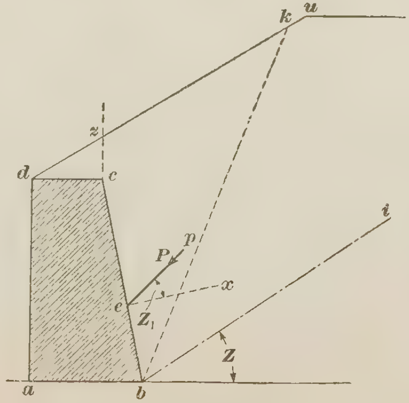
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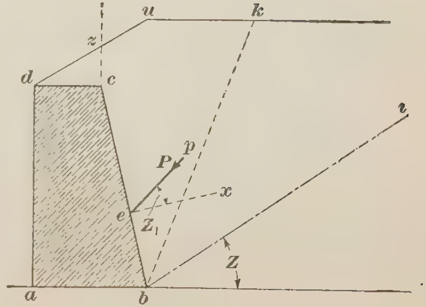
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(f)

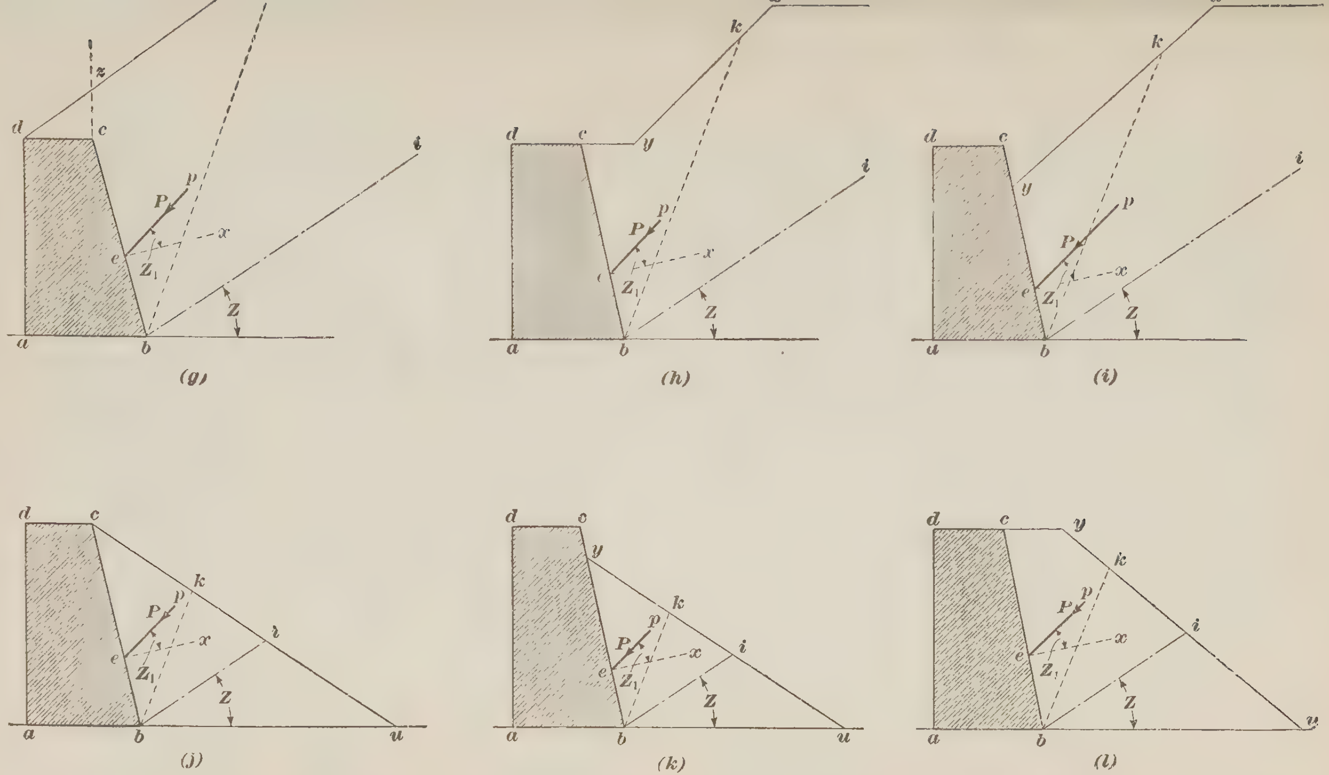

FIG. 16

its face with the slope of maximum pressure, as at (*d*) and (*f*), or because the surcharge runs over the top of the wall, as at (*e*), (*f*), and (*g*), or by the backing sloping down from a point behind the back of the wall, as at (*l*)—the point of application of the resultant pressure will be raised, because the center of gravity of the mass of backing causing the maximum pressure will lie farther from the slope of maximum pressure. The one possible exception to this rule, where the cross-section has a perimeter of more than three sides, is when it contains a reentrant angle, as *y*, Fig. 16 (*h*).

It will be observed that the effect of lowering the level of the top of a surcharge, and so reducing the amount of backing—as at (*d*) compared with (*c*), and at (*f*) compared with (*e*)—or the removal of some of the backing when normally level—as at (*l*) compared with (*a*)—raises the point of application of the pressure, and so increases its leverage; but, as the amount of the pressure is at the same time reduced to more than offset this increase of leverage, the moment of the overturning pressure is reduced.

42. Battered Face.—According to Moseley's theory, the use of a battered face on a retaining wall has no effect on the overturning moment, but modifies the resistance offered by the wall in the same manner as explained in Art. **29** in connection with Coulomb's theory, increasing slightly the required base, but decreasing materially the average thickness and total quantity of masonry.

43. General Method of Procedure.—In what follows, the preceding principles will be combined into a general statement indicating the successive steps that should be taken in the analysis of a retaining wall according to Moseley's theory.

Fig. 17 represents a surcharged wall *a b c d* with the toe of the surcharge lying at the outer face *d* of the top of the wall; the description and directions, however, will apply equally well, in substance, to other conditions. The wall may be stepped, as indicated by the dotted lines, but this does not alter the principles involved.

First lay off from the heel of the wall the slope of repose *b i* of the material of which the backing is made. Usually, the slope of *b i* is assumed as 1½ horizontal to 1 vertical, as this is approximately the worst slope to be expected with the materials commonly used for filling. The slope of maximum pressure *b k* for any material and condition lies midway between the slope of repose *b i* and the vertical plane *b t* passing through the heel of the wall; that is, *b k* bisects *t b i*.

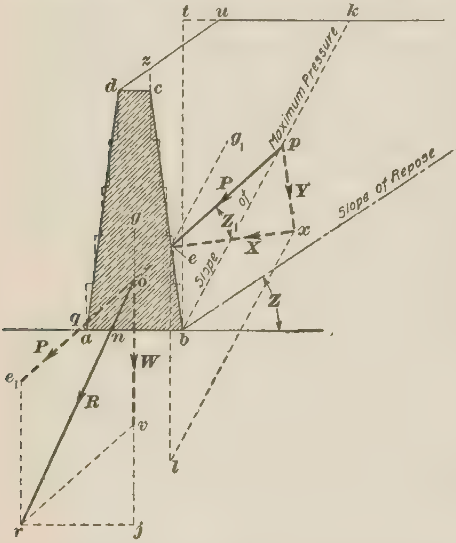

FIG. 17

Considering, now, a section of wall and backing contained in one unit of length measured parallel to the line of the wall, ascertain the weight and center of gravity g_1 of the backing lying between the slope of maximum pressure and the back *b c* of the wall, with an extension *c z* in a vertical plane above the top of the back of wall, to the slope of the surcharged filling; that is, of the mass represented in cross-section by tne polygon *k b c z u k*. To this mass should be added the weight of any load that may rest on the filling. From the center of gravity of this portion of the backing (with its superimposed load, if any), draw a line $g_1 e$ parallel to the slope *k b* of maximum pressure, until it intersects the back of the wall at *e*, which represents the center of pressure of the. backing—that is, the point of application of the resultant pressure. Through *e* draw *e x* perpendicular to the back of the wall (or to the plane passing through the back of the wall at its top and bottom, if the back of the wall is stepped), and also a vertical line, on which scale off a length $e\,l$ to represent the weight of the filling, with its superimposed load, if there is any. Draw $l\,x$ parallel to $b\,k$. Then, $e\,x$ will represent the component X of the pressure P perpendicular to the back of the wall. Draw $x\,p$ parallel to the back of the wall, and $e\,p$ making with $e\,x$ an angle Z_1 equal to the angle of friction between the filling and the back of the wall (usually taken as 33°, which corresponds to a slope of $1\frac{1}{2}$ to 1). Then, $p\,x$ will represent the magnitude and direction of the friction Y that may be developed by the pressure $e\,x$ of the filling on the back of the wall; and $p\,e$ will represent the resultant pressure P. Producing $p\,e$ and dropping a perpendicular $q\,a$ to it from the toe a of the wall, the lever arm of P with respect to a is determined. The product of $p\,e$ (in units of weight or pressure) and $q\,a$ (linear units) represents the overturning moment.

Now find the weight W and the center of gravity g of the wall $a\,b\,c\,d$, together with the small amount of filling $d\,c\,z$ resting on its top (and also of any other load or force resting on the wall, if there is any), and from this center of gravity draw the vertical $g\,j$. From the point o where this vertical meets the line of action of P, lay off $o\,v = W$, and $o\,e_1 = P$; complete the parallelogram $e_1\,o\,v\,r$, and draw the diagonal $o\,r$, which represents the resultant pressure R on the base or foundation of the wall; this resultant intersects the base at n. Under ordinary conditions, $a\,n$ should not be less than $\frac{1}{3}\,a\,b$.

The resultant total pressure $o\,r$ may be resolved into a vertical component $o\,j$ and a horizontal component $r\,j$; the latter thrust must not exceed the product of the former and the coefficient of friction of the wall on the base (or the angle $r\,o\,j$ must not exceed the angle of this friction), unless there is some other force that helps to resist the tendency to slide.

EMPIRICAL RULES

44. As previously stated, none of the mathematical theories of the equilibrium and stability of retaining walls is altogether satisfactory, because of the many and important elements of uncertainty entering into them. But, by the use of the best theories available, combined with the results of experience and observation, very simple rules have been formulated that probably are a more correct guide than even the most perfect theory yet advanced. A thorough understanding of these theories is, however, very important, both because they are often used or referred to, and also because they give a general idea at least of the conditions that should be taken into account.

Probably the best series of empirical rules is that presented by John C. Trautwine in his well-known *Civil Engineers' Pocketbook*. These rules are given, with slight modifications, in the following articles.

45. Vertical Walls.—For a vertical wall resting on a foundation of masonry suitably enlarged for a proper distribution of the load on the soil, with the top of the fill leveled off at the top of the wall, and with the backing deposited loosely, as when dumped from carts, the ratio of the thickness to the height of the wall should be .35 for a wall of cut stone, or of first-class large-ranged rubble, in mortar, or of concrete; .4 for a wall of good common rubble or brick, in mortar; and .5 for a wall of dry rubble.

If the backing is deposited in layers well compacted, the thickness may be slightly reduced, without sacrificing the stability of the wall. It is not, however, customary to reduce the thickness on this account; for, although specifications frequently call for the filling to be so deposited and compacted, this is rarely done, and an engineer attempting to enforce such specifications will be likely to receive more abuse than satisfaction.

46. Battered or Stepped Back.—For a wall with a battered or stepped back, Trautwine recommends using the same *average* thickness as for a vertical wall, increasing the base by the same amount that the top width is decreased. As seen in the discussion of Moseley's theory, this will give a small increase in the stability over that obtained from the vertical wall, while the volume of masonry remains the same. The reduction in average thickness can only be very slight without decreasing the stability, so that it is ordinarily disregarded.

47. Battered Face. A wall with a battered face may be made to give the same stability with a materially smaller volume and average thickness that would be required if a vertical wall were used.

For equal stability, a wall of triangular cross-section with a vertical back and battered face should have a base 1.225 times as great as the thickness of a vertical wall, as may be seen by reference to Fig. 18, in which $a\,b\,c\,d$ represents a vertical retaining wall, and $a_1\,b\,c$ a wall of triangular section having the same height and moment of stability: g and g_1 represent their respective centers of gravity, and $m\,a$ and $m_1\,a_1$ the lever arms of their weights about their respective toes.

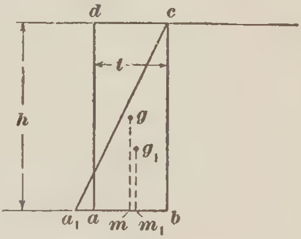

FIG. 18

Let h = height of wall;

t = thickness of vertical wall;

b = width of base of triangular wall;

w = weight of masonry per unit of volume.

Then, in a portion of the wall one unit in length, the weight of the vertical wall $a\,b\,c\,d$ will be $h\,t\,w$, and its moment of stability,

$$h\,t\,w \times \frac{t}{2} = \frac{h\,t^2\,w}{2}$$

For the same length, the weight of the triangular wall will be $\frac{b}{2}h\,w$, and its moment of stability,

$$\frac{b}{2}h\,w \times m_1\,a_1 = \frac{b}{2}h\,w \times \tfrac{2}{3}b = \frac{h\,b^2\,w}{3}$$

In order that these two walls may have the same degree of stability, we must have

$$\frac{h\,t^2\,w}{2} = \frac{h\,b^2\,w}{3};$$

whence $$b = t\sqrt{\tfrac{3}{2}} = 1.225\,t$$

The amount of masonry in the triangular wall of equal stability against overturning will therefore be only .6125 times that of the vertical wall. Such a wall will, however, have serious objections, as will be explained further on.

In a similar manner, the dimensions of a wall having a vertical back, a given batter of face, and a given stability may be computed.

48. Surcharged Walls.—As indicated in Art. **39,** the pressure on the back of a surcharged wall is greater than if the backing were level with the top of the wall, and the thickness of the wall must be correspondingly greater.

When the surcharge runs over the top of the wall—as in Fig. 17, and in Fig. 16 (*e*), (*f*), and (*g*)—there is a slight increase in the weight resisting overturning by the addition of the triangle of earth *dcz*, Fig. 17, as well as the larger increase in the weight of the wedge of backing pressing against the back of the wall. For a height of surcharge less than about a quarter of the height of the wall, the additional weight of the filling resting on the top of the wall will offset the extra weight of the overturning wedge; but, as the height of the surcharge increases with the slope running over the top of the wall, the overturning pressure increases rapidly, while the increased resistance due to the earth resting on the top of the wall changes only slightly with the increase in thickness of the wall. Thus, it is readily seen that, if the surcharge is large and is allowed to run over the top, the thickness of wall required will be greater than if the surcharge slope started at the back of the wall. Table III shows the proper ratios of thickness to height for vertical walls with various amounts of surcharge. After ascertaining the thickness of the vertical wall that would be required for restraining a surcharge bank, the form of the wall may be altered to give a battered face or back, or both, in the same way as though the top of the backing were level with the top of the wall (see Arts. **46** and **47**).

It will be noticed that Table III calls for an increase in the thickness of the wall for the increased height of the surcharge, far beyond the .55 times the height of the wall at which the "plane of maximum pressure" will usually cut the plane of natural slope with the toe of surcharge at the back of the top of the wall. This is because the results of experiment and experience show that the pressure actually does increase beyond the limit indicated by the best theories that have yet been formulated. Plainly, some important factor is omitted in these theories.

TABLE III

SURCHARGED VERTICAL WALLS—RATIO OF THICKNESS TO HEIGHT

Ratio of Surcharge to Height of Wall	Toe of Slope at Back of Wall			Toe of Slope at Front of Wall		
	Cut Stone	Mortar, Rubble, or Brick	Dry Rubble	Cut Stone	Mortar, Rubble, or Brick	Dry Rubble
.0	.35	.40	.50	.35	.40	.50
.1	.42	.47	.57	.42	.47	.57
.2	.46	.51	.61	.46	.51	.61
.3	.49	.54	.64	.49	.55	.66
.4	.51	.56	.66	.53	.60	.72
.5	.52	.57	.67	.58	.65	.79
.6	.54	.59	.69	.62	.70	.85
.7	.55	.60	.70	.65	.74	.91
.8	.56	.61	.71	.67	.77	.96
.9	.57	.62	.72	.69	.80	1.00
1.0	.58	.63	.73	.71	.82	1.04
2.0	.62	.67	.77	.81	.96	1.26
3.0	.63	.68	.78	.85	1.02	1.35
5.0	.64	.69	.79	.88	1.07	1.44
25.0	.68	.73	.83	.92	1.11	1.50

SETTLEMENT OR CRUSHING OF THE TOE

49. It should be noted that even the hardest stone is compressible and may be crushed, and that the subfoundation is generally even less rigid and strong. There is, consequently, a greater or less yielding of wall and subfoundation caused by the pressure of the backing; and the greater the pressure, the greater will be the yielding. As the pressure on the base of a retaining wall is greater at the toe than at the heel, this yielding is also greatest at the toe. With a well-designed wall resting on a firm subfoundation, this yielding is so slight as to be negligible, and generally hardly perceptible even with the most careful observations; but, when the subfoundation is soft and the center of pressure lies very near the toe of the wall, the yielding may become of great importance.

50. The destruction of the toe of a retaining wall decreases the lever arm of the weight, and, therefore, the moment of stability of the wall. It increases also the intensity of pressure on the subfoundation, by reducing the width of base.

51. By the settling of the toe, the wall leans forwards, thereby throwing its center of gravity more nearly over the toe, thus further reducing the lever arm of the weight and the moment of stability of the wall. This also increases the pressure on the toe, tending to increase (and more rapidly) the settlement there, and speedily leading to the downfall of the wall.

52. The settlement at the toe and the resulting leaning forwards of the wall increase the pressure of the backing, by increasing the amount and weight of the filling lying between the back of the wall and the slope of maximum pressure, and by enlarging the wedging angle through which the pressure tends to throw the wall forwards. As the wall yields to this increased pressure, the pressure still further increases.

53. By the settlement or crushing of the toe, the wall leans forwards; the beds become inclined, and the tendency of the wall to slide as a whole on its base, and of the different beds to slide on each other, is increased. After a sliding motion has begun, the rougher surfaces are made smooth by attrition, the friction decreases, and further motion occurs, gradually leading to the complete destruction of the structure.

54. The foregoing considerations will show the importance of taking all necessary measures to prevent settlement or crushing of the toe. Where the resultant pressure passes through the center of the base, the settlement is uniform, and does not cause serious harm; the nearer the resultant pressure is to the toe, the more uneven the settlement is likely to be; hence the importance, especially in soft soils, of making the eccentricity of the pressure as small as possible, or of providing for contingencies and uncertain conditions by the use of a large factor of safety.

CONSTRUCTION

MATERIALS USED

55. All classes of masonry materials—stone, brick, and plain and reinforced concrete—are used for the construction of retaining walls; and all grades are used according to the importance of the structure. In general, for important structures, a very good quality is employed and is really most economical, for the saving of quantity nearly offsets the difference in cost, leaving very little expense properly chargeable to the improved appearance and greater durability. Timber and steel bulkheads may also be classed with retaining walls, although their stability does not generally depend on their weight alone.

STONE AND BRICK

56. Stone.—Though dry rubble is sometimes used for retaining walls, and may show very good durability, the more common specifications call for either cut stone or a good quality of rubble laid in mortar; and, as will be seen by referring to Art. 45 and Table III, a very noticeable saving of masonry is considered a proper allowance for the use of the better quality. Referring to such better class of masonry, a few of the generally accepted requisites will be stated, allowing sufficient range to cover all that class for which the rule in Art. 45 requires only .35 of the height for the thickness of a vertical wall.

All stone should be sound and durable, free from seams, dries, shakes, and flaws, of large size, and laid on its natural bed in Portland-cement mortar, in horizontal courses not less than 12 inches thick. According to the appearance desired, these courses may be specified to be regular or broken. Breaks in courses should not be allowed more frequently than one break to 50 square feet of face. The whole wall should be well bonded together; not less than one-fifth of the face of the wall should consist of headers extending either entirely through the wall or to a depth of at least 4 feet. Headers should be evenly distributed throughout the structure, and rest on, and be covered by, stretchers so as to thoroughly bond the whole together. No stone (except raisers used to break courses) should have a less width than height, or a length less than two and one-half times its height.

All joints on the face of the wall should break not less than 9 inches, and all stone should be dressed to lay for the full bed, and for not less than 9 inches in from the face in the vertical joints with $\frac{1}{2}$-inch to 1-inch joints. Sometimes, joints $\frac{1}{4}$ inch wide are specified for retaining walls, but they greatly increase the cost, while actually reducing the strength, as the stone has to be dressed, and does not have so strong a grip on the mortar as does the rough stone.

57. The backing should be in courses of the same thickness as the face, and, with respect to headers and size of stone, should be as specified for the face. Less care, however, may be taken with the vertical joints, and the back may be left rough, except to within 4 feet of the top of the wall, which should be dressed smooth. All stone in the heart of the wall should conform in size to the dimensions specified for the face, except as modified by necessity, because less room is left between the face stone and the back stones. No space wider than 6 inches should be left to be filled with spalls, which should be laid in mortar; no spalls should be allowed in the bed joints.

58. All joints in the face should be dressed to a line and in the specified plane, and the rock face between joints should nowhere project more than 4 inches beyond the neat lines. The foundation below 1 foot under the ground surface may be built of stone of the same quality as specified for the backing, or even of a little poorer quality for the back portion. The coping course should be of selected stone, running the full width of the top of the wall, and projecting on the face from 2 to 6 inches beyond the neat line, and of a length suitable for stretchers; the top should be dressed to give no depression greater than $\frac{1}{2}$ inch below a true plane resting on the top.

59. Brick.—Brick may be used for small retaining walls, but is an unsatisfactory material for large walls. A greater thickness will be required for a brick wall than for one of a good quality of stone, or of concrete. A brick wall exposed to the weather should be capped with stone.

CONCRETE

60. Concrete, either plain or reinforced, is an excellent material for the construction of retaining walls, and one whose popularity is deservedly growing. No inferior material should be used for concrete retaining walls, and great care should be taken to get a compact body with a smooth face; a wet concrete is to be preferred, as it is less likely to contain voids, and will also yield a better surface. The surface may be improved by greasing or soaping the inside of the form, and should be made richer than is necessary for the interior of the mass. This may be accomplished in either of two ways, both of which give very satisfactory results if well done, but are very unsatisfactory if poorly done.

61. In the first method, strips of sheet iron are provided, of a width a little greater than the thickness of the layers of concrete; before each layer of concrete is laid, the strips are placed 2 or 3 inches from the inside of the form, and this space is filled with the rich mortar, after which the layer of concrete is deposited to fill nearly to the top of the sheet-iron strips. The sheet is then raised nearly clear of the mass, and the concrete tamped to form a bond between the concrete and the mortar facing. The same operations are repeated for each course.

With proper care, this method gives very satisfactory results, but it is tedious and expensive, and generally the necessary care can be secured only by constant watching. The strips may be made with handles attached, or with holes through which hooks may be inserted, or the upper edge may be bent to the form indicated in Fig. 19, to aid in drawing them up. The sections should not be made so long that they will be difficult to handle.

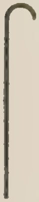
Fig. 19

62. A much less expensive, and fairly satisfactory, substitute for the foregoing is by "spading a face." In this method, the concrete is first laid full against the form; then the flat blade of a spade is run down the inside of the form and the handle worked back and forth so as to press the concrete away from the form. The cement, being lighter than the sand and stone, will flow back more readily, and, separating from them, will go to enrich the surface. The loss to the mass is but slight, while the gain to the surface is important. In order that this method may be satisfactory, the concrete must be wet enough for the mortar to flow back against the form after the spade is removed. The movement of the spade must be such that at different instants the concrete will be thrown back for the full height of the course; to do this properly, the blade should be placed flat against the form, as shown in Fig. 20 (*a*), and rocked back and forth, as shown at (*b*) and (*c*), so as to crowd in the concrete both at the top and at the bottom of the blade. Moreover, to produce satisfactory results at the middle of the course, the spade should be inserted to a less depth than the full length of the blade, as well as for its full depth.

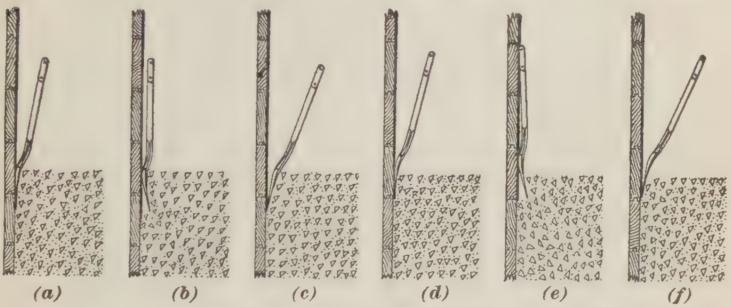

FIG. 20

A clearer idea of the movements may be obtained by reference to Fig. 20. A little practice will enable a workman to become very proficient, both as to speed and satisfactory results, working at three or four depths of penetration.

As this method is so much less expensive, and interferes so little with the rapid laying of the concrete, much less difficulty will be experienced in getting the work done according to specifications if spading is called for than if the separate layers of mortar were demanded. A face more like stone may be obtained by roughening the surface with a bush hammer or a similar tool.

63. Large stones may be thrown in with the concrete to advantage, both in economy and strength, when suitable stone is easily available. Some care should be taken when such stones are embedded, to avoid their coming in contact or forming large cleavage surfaces, for generally cement does not adhere to the surface of the stone as strongly as it coheres to other particles of cement (with some limestones the case is the reverse). Concrete with large stones so embedded is called **rubble concrete** or **concrete rubble,** according to whether the concrete or the large stones predominate.

64. Reinforced Concrete.—By the use of reinforced concrete, a large saving of material may be accomplished; for, with the encased steel to take tension, thus efficiently tying the different parts together, a hollow shell may be used, and the weight of the backing may be utilized to oppose its own overturning pressure. The saving in cost by the use of the reinforcement will be much less than the decrease in volume, but will still be considerable in a wall of large size.

Fig. 21 shows a retaining wall of reinforced concrete. In the figure, (a) is the elevation of the face of the wall and (b) is a cross-section on pq; a is the top, b the face, and c the base of the wall. The base is much wider than the thickness of the face; they are connected at frequent intervals by counterforts d that slope from the top of the wall to the inside line of the base. The face, base, and counterforts are reinforced with steel rods e to resist any tension and to prevent surface cracks. This is a very economical type of retaining wall, as the weight of earth that rests on the inner or back part fg of the base helps to maintain the stability; less weight of masonry is therefore required.

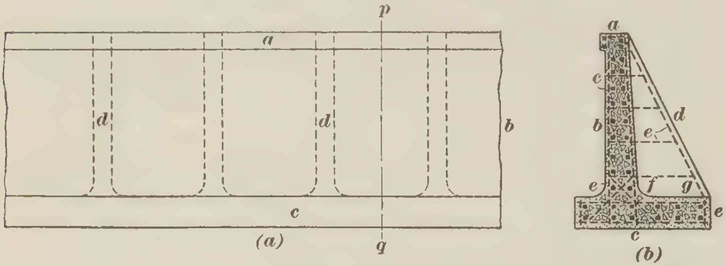

Fig. 21

65. Forms.—The forms for concrete retaining walls are generally of such size and extend to such height above the ground surface that it is not practicable to brace them efficiently from without, so that ties through the wall become necessary. Plain rods with screw ends and nuts are frequently used, and the ends are cut off a little within the plane of the surface of the concrete after the forms are removed. Some engineers put the rods through pipes that are 2 or 3 inches shorter than the thickness of the concrete; when the form is removed, the rod may be drawn out and the pipe left in place, by simply breaking the concrete loose from the rod where not covered by the pipe. Telegraph or similar wire is also used, the ends being tied together and then drawn taut by inserting plugs and twisting; several strands of wire may be used if required, and the wires may be cut off just inside the planes or faces of the wall. After cutting the rods or wires, or withdrawing the rods from within the pipes, the holes left in the face of the concrete may be plastered up. Fig. 22 illustrates both rod and wire

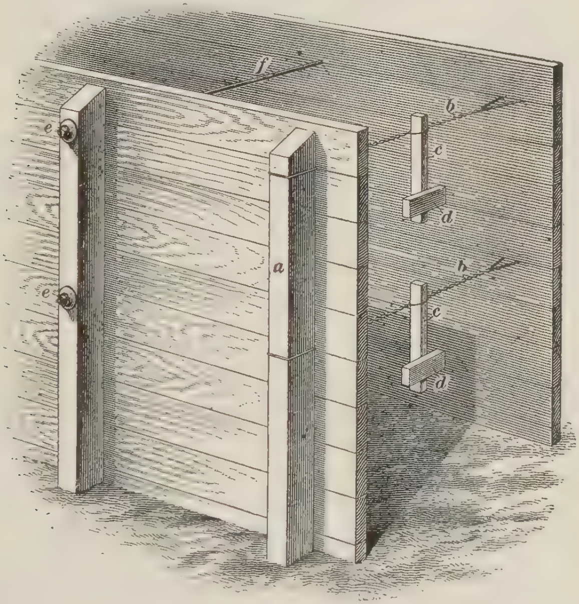

Fig. 22

ties; the rods are shown at *f*, the ends being held by nuts and washers *e*, *e*; the wires are shown at *b*, *b*, the ends being looped around the posts *a*. The sticks *c* and *d* are for the purpose of twisting the wire so as to bring the sides to the proper distance apart and hold them there.

66. Before the concrete is placed in the forms, braces may be used on the inside to hold the sides out to their proper place, which is as important as the duty of the rods or wires to prevent their spreading too much. As the work progresses, these braces may be removed and the concrete may be relied on for this duty.

FOUNDATION

67. A good foundation for a retaining wall is as important as that the wall itself should be properly designed and built. *Nearly all the failures of retaining walls are due to poor foundations and improper design of the base of the wall.* Before the wall is built, a thorough examination of the character of the soil should be made, as explained in *Foundations*, Part 1. It should be borne in mind that sand, especially quicksand, is, as a rule, a very treacherous and unreliable material. Both sand and gravel, however, are excellent mixing materials for increasing the bearing power of soft clays and loam.

Generally, foundations have to be carried to a considerable depth below the ground surface, so as to avoid the dangers of frost and scour. In the northern United States, 4 feet below the surface is a common minimum requirement. Often, this is also of assistance in maintaining the stability of a retaining wall against sliding, as the coefficient of friction of the masonry on many soils is much less than that of two courses of masonry on each other—especially if the courses are laid in mortar, which binds them together. Considerable assistance in resisting sliding is gained by a few feet depth of earth in front of the walls above the bottom of the masonry.

68. On account of the greater pressure at the toe of a retaining wall, it is customary and advisable to enlarge the footing course by offsetting at the front below the ground level, as shown in Fig. 23 (*a*). On soft soil, this offsetting may be made of considerable length by making the depth below the surface greater than would be otherwise required. The offsetting may be made by a succession of steps, as in Fig. 23 (*b*), each with a rise about equal to the offset. With strong stone or other good material, the spread may be at a greater rate than 1 to 1, while with poor material it should be less. Every inch of such offset at the front of the foundation is a valuable aid to the stability of the wall as to overturning, and might well be taken advantage of to a greater extent than is customary.

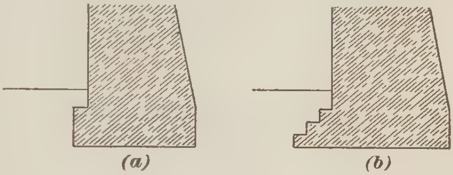
(*a*) (*b*)

FIG. 23

Walls are sometimes built with a similar offset at the heel; but this is of little use, simply adding a little to the weight and slightly to the leverage going to make up the moment of stability; while the same offset at the toe increases this moment several times as much.

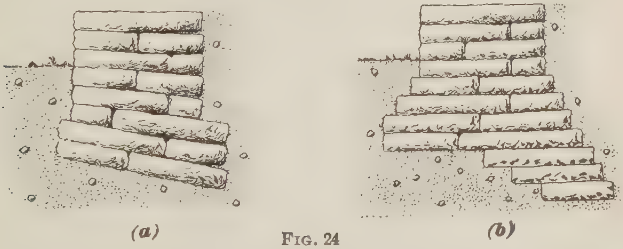
(*a*) (*b*)

FIG. 24

69. To prevent sliding, the footing of a retaining wall is sometimes built on an incline, as indicated in Fig. 24 (*a*). Generally, this is of doubtful value, because of the compressibility of the soil. The inclined beds are objectionable, because of the facility with which moisture gathers in them. The design shown in Fig. 24 (*b*) is a better one.

70. As the toe of a retaining wall receives so great a load, piles are sometimes used to relieve the soil at this point (see Fig. 25). At best, this is an unsatisfactory arrangement, because by it, while the back edge of the wall rests on the soil near the natural surface, the support of the front is transferred to a lower level, and presumably a harder stratum of soil. If the upper soil is as soft as this expedient would suggest, there will be danger of settlement at the back, while the front is kept up to its original place, and the result may be as indicated in Fig. 26, followed by the collapse of the wall. If the soil is sufficiently firm to prevent such settlement of the back, an offset at the toe can generally be more easily provided, and will be more efficient than piles. Piling should never be used except as a temporary expedient, unless the timber keeps constantly wet; otherwise, it will rot. When the piling keeps constantly moist, it may be used under the full width of the wall to give a foundation in soft soil, in which case it is well to give greater support to the front than to the back of the wall.

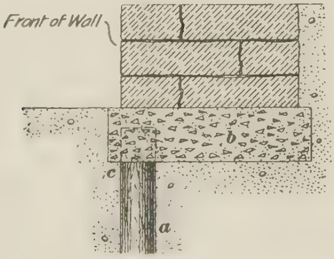

FIG. 25

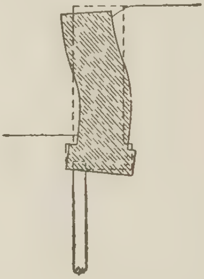

FIG. 26

OTHER DETAILS

71. Coping.—The top of a wall, being more exposed to injury from the weather and other destructive agencies, should be made of carefully selected stone or of a richer concrete than is required for the body of the wall. This upper course or part of the wall is called the **coping** or

cap, and is generally given a width of about 2 or 3 feet. The coping should, if practicable, project from 2 to 6 inches (usually 3 or 4 inches) beyond the body of the wall, so that the drip will clear the face of the wall. If of stone, the coping should be made of large stone, of the full width of the course, and with as few transverse joints as possible. The top is frequently built with a slope to favor drainage.

72. Batter of the Face.—It has been explained that, for resistance to overturning, economy of quantity favors the use of a heavy batter for the face of retaining walls, even to the extent of using a triangular cross-section of wall. There are, however, serious objections to this form of construction: the earth near the surface is exposed to disturbing influences (chiefly frost in cold climates), which are more destructive to the stability of structures resting on, or otherwise brought in contact with these top strata, than to those either above ground or at a lower depth; for this reason, as well as from the natural objection to an acute angle or thin section, the top of the wall is never reduced to such an extent.

Aside from the objection to the thin top, an excessive batter exposes the face of the wall to undue destructive influences. With a vertical or slightly battered face, the rain or drainage from above passing over the face will run off rapidly with no injury to the wall; but with the greater slope, there is a greater inclination for the water to soak into the joints and seams, and do greater or less injury according to the quality of the construction, and according to whether freezing follows. Seeds, also, lodge in the joints and the resulting vegetation frequently causes serious damage to the wall.

Another serious objection to a large batter that frequently exists is the reduction of width available for use at the top of the wall.

73. Back of Wall.—The back of a retaining wall may preferably be left rough, thus increasing the friction of the filling. As an exception to this, the top part of the back (down to the frost line) should be left smooth, and preferably with a forward slope, so that the frost and other surface influences will not cause any upheaval.

74. Bond.—Stone retaining walls should always be thoroughly bonded together, both transversely and longitudinally, so that the wall may act as a unit in resisting the thrust of the backing, instead of as a number of smaller units. The front part should be made of large and sound stone, not only for the sake of appearance, but also because the front is most strongly affected by the weather.

75. Drainage.—Most soils have a smaller angle of repose, and consequently cause a greater pressure on the wall, when thoroughly dry than when slightly damp; but, if saturated with water, they become not only heavier, but semifluid, and cause a much greater pressure. If a large retaining wall in a wet place is carried down to a subfoundation on impervious material, the escape of the water in the backing may be cut off and such an increase of pressure may follow. Care must be taken to avoid this danger by proper drainage, which may be effected by openings, called **weep holes**, through the wall. Weep holes are also sometimes provided to keep the water out of joints and seams in the wall itself.

76. Sea Walls.—As the earth behind a sea wall is generally saturated, it is heavier than the same filling would be if dry, and also naturally assumes a flatter slope (perhaps even nearly level); its pressure on the wall is greater and requires a greater thickness of masonry than would be necessary for an ordinary retaining wall.

77. Buttresses.—Buttresses are comparatively costly, and are seldom justified in the construction of ordinary retaining walls. For a very high wall, however, if economy of construction is the all-important factor, they may be used to advantage. As the extra cost per unit of volume is chiefly in the forms when concrete is used, a less height of wall may favor the use of buttresses with this material than where stone must be cut for all the extra angles of the buttresses.

The difficulty of satisfactorily bonding a buttress to the main wall, and generally the unsightliness and waste of room, add to the reasons for their rare use. Buttresses are sometimes used to stiffen old walls that have shown signs of impending failure.

78. Relieving Arches.—Masonry may sometimes be saved in retaining walls by the use of **relieving arches** (see Fig. 27). In this design, a series of transverse arches carry the earth, while the face wall has very little or no load, and is of use chiefly to close the openings and give an appearance of an ordinary retaining wall. The structure as a whole must be able, with the aid of the earth lying inside the arches, to bear the pressure of the earth on the back; as the weight of the structure is less than if the arches were filled, the width of the base must be materially greater than would be required for an ordinary wall.

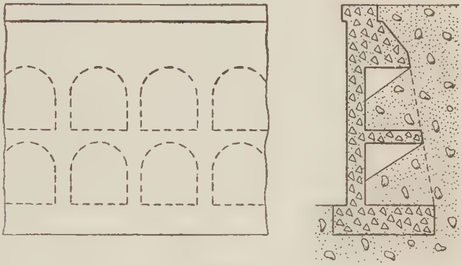

FIG. 27

Generally, the great expense of these arches far outweighs the saving of the material; but with the use of concrete, they may occasionally be advantageously employed for high walls.

CULVERTS

INTRODUCTION

1. Conditions to be Met.—Wherever the contour of the ground is such that the water even from a single storm will be dammed up by the embankment of a road, a source of danger to the bank is created, even if there is ordinarily no stream at the place. The magnitude of this danger varies with the nature of the material of which the embankment is made, with the amount and depth of the water, and especially with the relation of that depth to the height of the bank. If the water flows over the top of the embankment, the latter will quickly be washed away; and even at a considerably less depth, the material of the bank may become so softened by the water that it will have little strength to resist the pressure of the water, and its usefulness will be otherwise seriously impaired. In addition to the danger of destruction of the embankment, there may be damage done by flooding adjacent lands; and, in general, abutters are not slow to take advantage of the slightest occasion for a claim of damage against a railroad company.

To provide against these dangers, some way must be prepared for the passage of the water across the embankment. Where the possible volume of water is large, a bridge is usually necessary; where the volume is small, a *culvert* is sufficient.

2. Definition.—A **culvert** is essentially a small bridged passage for the conveyance of water across the line of a railroad or other way.

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3. Location of Culvert.—The natural location of a culvert is usually at the lowest part of the embankment in which the water tends to collect. This is generally also the widest part of the embankment. Frequently, a culvert may be located at a slightly higher point, allowing the water to collect to a small depth before being carried away by the culvert, it being expected that the silt and débris brought down by the stream will soon raise the level of the ground to that of the bottom of the culvert. This will allow the use of a shorter culvert than would be required if the culvert were constructed at the lowest point; besides, a site may be selected that affords a better foundation.

4. Care must be exercised in the location of a culvert, lest the safety of the bank or other property be endangered, and lest opportunity be given the abutters for damage claims, either for flooding their land or for changing its surface, or possibly for the diversion of a natural stream. This danger of damage claims is not confined to the up-stream side of the bank, but is equally likely to exist on the lower side. Neither is the danger of such claims (which are upheld by the courts) necessarily governed by the actual damage done, for common law gives landowners a right to allow no interference with the processes of nature so far as it may affect their land. Actually, it might be a matter of indifference or even an advantage to an abutter to have the channel moved 50 feet to one side or the other, or to have soil deposited on or taken from his land; but if he objects to these alterations of natural conditions, his claim for damages will hold. The engineer should be careful not to divert a stream from the land of one owner to that of another, lest the one consider himself injured by receiving what the other claims to be injured by losing. For these reasons, it is seldom advisable to change materially the natural location of a culvert. The culvert, however, may be made to cross the line of the road at right angles, instead of on a skew, thus saving considerable expense.

Easy approach to the inlet of the culvert is necessary, especially where the volume of water is likely to be large or the velocity great, in order that the channel may not become clogged or the water force a passage by some other way.

5. Culvert Avoided.—A culvert may sometimes be avoided altogether, or one culvert made to take the place of two, by excavating a ditch through a ridge separating two valleys. This plan is especially good where the general trend of the slope is parallel to the line of the road and is simply broken by a small spur or hill reaching just across the line.

6. Blind Drains.—Where the volume of water is very small, so that no danger to the bank or adjacent property need be feared, blind drains may be made by filling with loose rock, and allowing the water to percolate through the bank. These drains usually silt up, and soon become tight; but in the meantime the bank becomes more compact and less liable to injury.

CLASSIFICATION AND MATERIALS

TYPES OF CULVETS

7. Culverts are divided into four general classes; namely, *open*, *box*, *arch*, and *pipe* culverts.

8. Open culverts consist of abutments, with possibly one or more intervening piers, on which stringers or beams rest. These culverts, which are practically small bridges, are used where the depth of fill is insufficient to allow of a cover with filling over the top. The objections to them are the danger arising from their being open, and the interruption they cause in the smoothness of riding, on account of the fact that vehicles, in crossing them, pass from the ballast to a rigid support at the abutment, then to the elastic structure forming the bridge, then to another rigid support, and then to the ballast again. These successive changes are the more objectionable because they occur within a very short distance.

The abutments for open culverts are designed by the same rules as those for longer bridges, as explained in *Bridge*

Piers and Abutments. The stringers are designed as for small bridges.

9. **Box culverts,** Fig. 1, have side walls *W, W* supporting a horizontal covering *C* over which the fill is made. The bottom of the waterway *O* is usually paved. Box culverts are generally built of stone or wood, although concrete, usually reinforced with steel, is sometimes used.

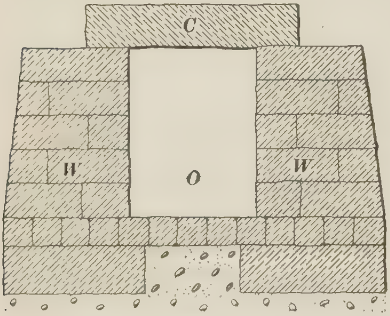


FIG. 1

10. **Arch culverts** are similar to box culverts, except that the horizontal roof is replaced by an arched cover, and the side walls are designed to resist the thrust from the arch. Arch culverts may be used for much larger openings than box culverts; in fact, the limit of practical construction with an arched roof is far beyond the limits within which the name culvert is generally used. At present, arch culverts are most frequently made of concrete, either plain or reinforced, although wood and rubble-stone masonry have been employed. Cut-stone masonry is also used for arch construction, but generally only for larger openings.

11. **Pipe culverts** are made by laying pipe either directly on the soil or in a bed of concrete. Pipes with as large a diameter as 5 feet have been used for this purpose. Pipe culverts are deservedly growing in popularity for small openings, because of the ease of construction, of the large amount of water they can convey, and of their small cost. They are generally made of cast iron or vitrified terra cotta.

12. **Multiple Culverts.**—The greater number of culverts have only one opening, and are known as **single culverts.** Frequently, however, a larger waterway is

required than would be practicable for a single opening, without changing the character of the structure; in that case, several walls are built to support the roof, and the space between each two walls forms an opening for the culvert. Such culverts are called **multiple culverts**, the special names **double** and **triple culverts** applying, respectively, to two and three openings.

MATERIALS

13. Terra Cotta.—Vitrified terra cotta is extensively used for pipe culverts, but is easily broken either by a blow or by a steady load. The breakage of pipe in transportation and handling is an important factor in the cost of culverts of this material. Being made in short lengths, the pipe yields readily to uneven settlement when laid on a soft foundation, thus reducing the breakage that might occur after laying if the lengths of pipe were as great as with cast-iron pipe. This uneven settlement, however, creates extra stresses of unknown magnitude. The settlement is also liable to open the joints or to cause water to collect in the low parts, where it may freeze and burst the pipe.

When terra-cotta pipe is used for culverts, it should be made thicker than would be necessary for sewer pipes, which are generally laid in trenches dug for the purpose, where little settlement is possible. Moreover, in sewer work the filling naturally arches over the top of the pipe, transferring much of its weight to the firm earth that has been undisturbed, and relieving the pipe accordingly. For culverts in a new fill, this relief is not to be expected.

14. Iron and Steel.—Cast iron is largely used for pipe culverts, and is superior to terra cotta, because it is stronger and less friable. With proper care in laying to prevent uneven settlement, cast iron makes a very durable culvert, provided that the depth of fill is sufficient to take up the impact from the trains. Wrought iron and steel have been used to some extent, but the corrosion of both these materials in the soil is rapid, and their use cannot be recommended.

Considerable settlement may take place with an iron-pipe culvert without danger to the structure, provided that there are no abrupt changes in the support given to the pipe. Iron pipe in 12-foot lengths is very convenient for the construction of small culverts.

15. Timber.—In districts where stone is scarce and timber plentiful, wood is often used in making culverts, generally in the single- or double-box form. Timber culverts, however, are usually but a temporary expedient, and are frequently replaced by pipe or concrete after the road is opened for traffic and the timber has begun to decay. A timber culvert may last from 4 to 12 years before it needs renewing, the period depending on the character of the soil and climate, as well as on the kind and condition of the timber used. To allow for the decay of the timber, 2 or 3 inches extra thickness of wood should be provided. Timber has been used to some extent for arch culverts, but the conditions favoring this construction are infrequent.

16. Stone Masonry.—Where good stone can be easily and economically obtained, it is the favorite material for culvert construction. Unless the span is too long, the use of this material naturally suggests the box form, either single or double. Generally, a very low grade of material and workmanship has been used for this construction, and often the stones are laid dry (without mortar). Although there is no reason for the use of first-class stone, dressed with the care desirable for bridge piers and abutments, the use of poor stone is likely to prove a false economy, for a large expense is involved in repairing a break in a culvert, especially if the bank is high. Stone with seams should be particularly avoided for the cover, on account of the transverse stresses to which the roof is subjected. No stone whatever should be used that is likely to disintegrate in water. A great many failures of stone box culverts have been due to disintegration, or to the use of seamy cover stones. If stratified stone, however faint the stratification may be, is used, it should be laid on its natural bed, with the strata horizontal, whether the stone is placed in the walls or in the cover.

17. With the present low prices of Portland cement, there is no real economy either in laying stone culverts dry or in using natural cement. Mortar composed of good material renders the structure, to some extent, monolithic, and therefore stronger; the whole structure thus becomes a unit instead of being made up of numerous units, each subject to settlement or stress without help from the adjacent units.

Although there are some natural cements that show good qualities, and under certain conditions may be as satisfactory as some brands of Portland cement, natural cement, being less uniform, is more uncertain, and a richer mortar should be used with it than with Portland cement. A 1 : 2 natural-cement mortar is commonly used as equivalent to a 1 : 3 Portland cement. Natural cement should never be used in freezing weather; good brands of Portland cement may be used, with proper precautions, in weather considerably below the freezing point. In such temperatures, or when such temperatures are expected before the mortar can set, salt should be added to the water with which the mortar is mixed. A rule that experience has proved satisfactory is to use 1 pound of salt to 18 gallons of water, for a temperature of 32° F., with an additional ounce of salt per 18 gallons for each degree below 32°. The proportion of salt should be based on the coldest weather expected between the time of mixing and the time of setting.

If the temperature is below freezing at the time of mixing, both the water and the sand should be warmed sufficiently to prevent all possibility of the freezing of the mixture before it is in place and past all danger of disturbance by the laying of stone on or near it. It is also important that the sand used should be clean, so that every grain of it will be entirely covered with cement.

CONCRETE

18. Plain Concrete.—Concrete, either plain or in combination with steel, is growing greatly in favor as a material for culvert construction, even frequently supplanting stone in regions where the natural conditions are especially favorable to the use of the latter material. Several causes have contributed to effect this change; namely, the low prices of cement and stone suitable for concrete, the ease and economy of transportation, the ease of handling and construction, and the fact that most of the work can be done by unskilled labor. To these advantages must be added the fact that concrete offers less resistance to the flow of water and catches less drift than a stone surface, thus materially increasing the capacity of the culvert for a given area.

What was said in Art. **17** regarding the selection of cement and sand for mortar, and the treatment of the ingredients in cold weather, applies with greater force to concrete culverts; for in stone masonry, if the cement is deficient, the structure will still have a greater or less stability, while with concrete the cement is the essential element, without which there would be nothing but a pile of stone and sand.

19. None but strong and hard stone should be used in making concrete for culverts, and it is preferable that the stone should be angular, because such form furnishes a better bond and better grip to the cement. "Crusher-run" (that is, stone broken to assorted sizes, just as it comes from the crusher, without screening) is more economical than stone of uniform size, and gives greater strength for the same nominal proportions, because the smaller stones partly fill the voids between the larger ones, leaving more cement to thoroughly coat the surfaces of the sand and stone, and make the bond more perfect.

Plain concrete lends itself readily to the arch form of culvert, for no cutting is required, as for stone. The cost is slightly increased by the price of forms; but, as they may be used repeatedly, the extra cost they involve is of comparatively little moment.

20. Reinforced Concrete.—For culverts, as well as for other structures subjected to bending stresses, reinforced concrete is preferable to plain concrete. In reinforcing the concrete with steel, the latter is so placed that it will resist the tension only, and the concrete the compression only, to which the structure is subjected.

The use of reinforced concrete permits of larger culvert openings than would be economical with either plain concrete or stone; so that a single larger opening may frequently take the place of the double culvert that might be the economical design with the use of either of the latter two materials.

Concrete is strongly alkaline, and so serves as a protector from the corrosive elements that tend to destroy the steel. Though, of course, greater safety is insured where there are no cracks in the concrete, so that no air will reach the steel, it is very probable that, unless the width of a crack is large compared with the depth, the air so reaching the steel will be attacked by the concrete and lose its corrosiveness.

21. Several styles of reinforced-concrete culverts have been built, both with arched and with flat roofs. With small spans, however, and with the usual depth of fill above, the possibility of great tensile stresses developing in an arch is so remote that reinforcement is not necessary in concrete arch culverts. Small rods, wire netting, and expanded metal are sometimes used for the purpose of preventing cracks in the surface. Longitudinal reinforcement in the invert may be conducive to economy and strength where the foundation is soft.

In flat-top or box culverts of reinforced concrete, the reinforcement consists of old rails or I beams, or of rods, wire netting, or expanded metal. Steel rods or bars are either employed in their usual form, or are twisted, corrugated, or in some other way prepared so as to form shoulders in the metal to give the concrete a firmer grip than that offered by the simple adhesion between the concrete and the metal. When reinforced concrete is used, the walls and pavement are also generally made with the same combination of materials. Plain-concrete walls are frequently combined with a reinforced-concrete cover.

22. Most of the methods of combining steel and concrete being patented, the engineer should make sure of his right to use the form of construction he wishes to adopt. Generally, this requires simply the purchase from the patentee of the form of steel to be used, the royalty for its use being included in the price of the steel. In some cases, a patent covers only the method of using simple materials to be purchased in the open market, and the right to employ that method must be obtained, by purchase or otherwise, from the patentee. Some patent claims conflict with one another, and the designer, in order to avoid trouble, should investigate the matter thoroughly.

GENERAL PRINCIPLES OF DESIGN AND CONSTRUCTION

SIZE OF WATERWAY

ELEMENTS DETERMINING THE SIZE OF WATERWAY

23. The size of the opening, or **waterway,** of a culvert is determined by the maximum amount of water that may pass through it in a given time, and also by the character of the culvert itself—its approaches and outlet.

The factors governing the volume of water that may pass through the culvert in a given time are: (1) *the drainage surface covered*, including: (*a*) its area, (*b*) its shape, and (*c*) its watercourses; (2) *the rate of rainfall:* (*a*) for a brief interval, and (*b*) for a longer period, also (*c*) the accumulation of snow; (3) *the character of the soil*, including: (*a*) the formation of the soil, and (*b*) the vegetation; (4) *the natural slope of:* (*a*) the bed of streams and rivulets contributing to the flow through the culvert, and (*b*) the surface of the land adjacent to these streams.

24. Drainage Surface.—With other factors unchanged, a larger culvert is required where a large than where a small drainage area is covered; but, for several reasons, the required waterway is by no means proportional to the area drained, even if other factors remain unchanged. In the first place, a larger rainfall per unit of area may be expected on a small surface than on a large one. In the second place, the conditions governing the speed of run-off being the same, it takes longer for water from the extremities of a large area to reach the culvert than from the extremities of a small area. Therefore, unless the storm is prolonged and remains of uniform intensity, the water from those parts of a large area that are near the culvert runs off before the water from the remote parts reaches the culvert. There will be less danger of the accumulation of water from a long and narrow drainage surface than from a surface of equal area but of more nearly square or circular form. In the third place, not all the rainfall is included in the surface discharge, and much that eventually reaches the culvert is retarded by being temporarily absorbed by the ground. This prolongs the time during which the storm water reaches the culvert, and reduces the amount to be discharged in a certain time. Finally, as water flows more freely when gathered in streams than when flowing over the surface of the ground or percolating through it, the water from a storm reaches the culvert sooner where the basin has many watercourses. The distribution of these watercourses in the basin should also be considered.

25. Rate of Rainfall.—In designing a culvert, provision must be made for the severest storms. In many localities, few data are available to indicate the maximum rainfall; and, where records have been kept, they frequently give only the daily precipitation, and usually cover comparatively few years. For a culvert draining a large area, the figures from these records may be a reasonable guide for determining the required waterway. It must be kept in mind, however, that the severity of a storm varies not only for different places within a given area, but at different times during the storm. Of a storm lasting a full day or more, one-half the rain may fall in one hour, or even in a few minutes.

26. In cold latitudes, the accumulation of snow that may be melted by a warm rain is of much importance in its relation to culverts. As the spring rains are frequently the most severe, and also release the accumulations of the winter's snow, it is generally at that season that a culvert is most taxed, especially if the surface of the drainage area is bare and frozen.

27. Character of Soil and Vegetation.—The nature of the soil, and the quantity and character of the vegetation in it, have a considerable effect on both the velocity and the amount of water that reaches the culvert. Rocky or porous soils without vegetation allow a larger and more rapid flow than clay or loam soils covered with vegetable growth.

28. Slope of Ground and Streams.—If the courses of the brooks and rivulets are rapid and the surface of the adjacent ground is steep, the flow of a stream and the velocity of its discharge are greatly increased. If the more distant parts of the valley are precipitous and open, while those nearer the culvert are flat and covered with vegetation, the rain falling on all parts will reach the culvert more nearly at the same time than if the ground in the neighborhood of the culvert is steep and otherwise adapted to easy and rapid flow.

29. Character of the Culvert and Immediate Surroundings.—The required waterway depends to a great extent on the character of the culvert itself and of its immediate surroundings. A steep grade in the culvert increases the discharge. The roughness of the inner surface of the culvert is another very important factor: the smoother that surface, the less is the danger that the culvert may be clogged with drift, and the less the resistance of friction, as well as the resistance caused by eddies. The position and shape of the inlet are also of importance; they should be such that the water will form no eddies on reaching the culvert. Of equal or greater importance is the outlet, for no more water can pass through a culvert than can run out of it.

30. If the water can be allowed to back up, so that it will flow through the culvert under a head, the volume that can be passed through a culvert of given size and character is greatly increased. It is not always possible, however, to allow this backing up of the water without danger of damage to adjacent property or to the embankment itself through which the culvert passes.

FORMULAS FOR SIZE OF WATERWAY

31. Introductory Remarks.—A theoretically perfect formula for determining the size and shape for the opening of a culvert would take account of all the factors mentioned in Art. **23**; but a knowledge of the effect of some of these factors is not yet sufficiently exact to make such formula possible.

It is not practically worth while to make an exhaustive study of all the conditions and records in order to reduce the size of opening to the minimum size that is consistent with safety. The conditions may be different in the future from those that have existed in the past, and the expense of an exhaustive study may exceed the extra cost of a more liberal design. It is seldom advisable to reduce the size to the very last inch given by formulas; for those formulas are but approximations founded on uncertain data and assumptions.

32. The formulas most commonly used for determining the size of culverts take account only of: (1) the area and form of the drainage surface; (2) the character of the soil, including vegetation; and (3) the slope of the ground. They are mainly empirical, and serve only as general guides.

Where sudden and severe storms are common or likely to occur, proper allowance must be made. In this, the engineer must use his judgment. Generally, very arid regions are so flat and their soil is so porous that no culverts whatever need be provided.

33. Myer's Formula.—The formula most generally used for determining the waterway for a culvert was constructed by E. T. D. Myer, and is as follows:

$$A = c\sqrt{M}$$

in which A = area, in square feet, of required opening;
M = area of drainage basin, in acres;
c = a coefficient varying with slope of ground, character of soil, and amount and character of vegetation.

The following are the values of c commonly used: $c = 4$ for mountainous and rocky ground without vegetation; $c = 1.5$ for hilly ground covered with verdure; and $c = 1$ for slightly rolling prairie.

Example.—What size of culvert with a rectangular opening is required for an outlet for a drainage basin 4,000 feet long and averaging 900 feet in width, the ground being hilly and under cultivation?

Solution.—Substituting in the above formula 1.5 for c and $\frac{4,000 \times 900}{43,560}$ for M,

$$A = 1.5\sqrt{\frac{4,000 \times 900}{43,560}} = 13.65 \text{ sq. ft.}$$

A box culvert $3\frac{1}{2}$ ft. × 4 ft., whose area is 14 sq. ft., might be used, but generally there is no intermediate size for box culverts between 3 ft. × 4 ft. and 4 ft. × 4 ft., so that a culvert with a 4-ft. square opening would be used in this case. Ans.

34. Talbot's Formula.—A later, and generally more satisfactory, formula has been derived by Prof. A. N. Talbot, of the University of Illinois. The significance of the symbols A, M, and c are as in Myer's formula, but Professor Talbot gives the following values for c *when used in his formula* (note that the substitution of these values for c in Myer's formula would not be permissible): $c = \frac{2}{3}$ to 1 for steep and rocky ground; $c = \frac{1}{3}$ for rolling agricultural country subject to floods at times of melting snow, and with the length of valley three or four times its width; $c = \frac{1}{5}$ to $\frac{1}{6}$ for districts not affected by accumulated snow, and where the length of the valley is several times the width.

Talbot's formula is as follows:

$$A = c\sqrt[4]{M^3}$$

Example.—What size of culvert would be required, according to Talbot's formula, for the same conditions as in the example in Art. **33**, with probabilities of floods at times of melting snow, and valley of moderate length?

Solution.—These conditions suggest a value of c, for use in this formula, of $\frac{1}{2}$. Substituting known values in the formula,

$$A = \tfrac{1}{2}\sqrt[4]{\left(\frac{4,000 \times 900}{43,560}\right)^3} = 13.7 \text{ sq. ft.}$$

This agrees closely with the result by the use of Myer's formula, and a 4-ft. square opening may be used.. Ans.

35. Comparison of Formulas.—Table I gives areas of waterway as computed both by Myer's and by Talbot's formulas:

TABLE I

Drainage Area Acres	Steep and Rocky Square Feet		Rolling Prairie Square Feet	
	Myer	Talbot	Myer	Talbot
1	4.00	.67 to 1.00	1.00	.17 to .33
10	12.65	3.75 to 5.62	3.16	.94 to 1.87
100	40.00	21.10 to 31.60	10.00	5.27 to 10.54
1,000	126.00	119.00 to 178.00	31.60	29.60 to 59.30
10,000	400.00	667.00 to 1,000.50	100.00	167.00 to 333.00

It will be noticed that Myer's formula gives larger openings when the drainage area is less than from $\frac{1}{6}$ to 2 square miles (according to whether the smaller or the larger coefficient is used with Talbot's formula), while for greater basins Talbot's formula gives larger results. In applying Myer's formula for large basins, considerable liberality should be used, especially if the upper part of the valley has steeper slopes than the part nearer the culvert.

For a general formula, Professor Talbot's seems more satisfactory and suited to a larger range of conditions than Myer's, but with small basins it would seem wise to provide a considerably larger opening than the formula calls for. This increase in size would be adopted as much because the increased cost is slight, compared with the increased safety, as because the figures seem unreasonably small. It should be borne in mind that both Myer's and Talbot's formulas are but rough approximations, and are intended, as Professor Talbot himself says, "more as a guide to the judgment than as working rules."

PRACTICAL METHOD

36. The waterway of most culverts is determined by the so-called "practical method," and not by any formula. This method consists in a study, made on the ground itself, of all the conditions, and a comparison with culverts built under similar conditions. An experienced engineer can in this manner estimate very closely the duty that the culvert is to perform, and decide on its dimensions accordingly.

In studying the ground, note should be taken of the past high-water marks, of the cross-section that the stream must have had at such times, and also of whether the current seems to have been rapid at the time of high water. Signs of erosion and drift are helps in this study of the ground; existing openings, if any, should also be noted, and the "oldest inhabitant" should be interviewed. All these signs (except the last) are liable to be underestimated; so a liberal allowance should be added to what they would seem to indicate. If the line is through new country, it should be kept in mind that, as the region is built up and forest and other vegetation is decreased, a larger and more rapid run-off is to be expected.

In designing culverts, it is well to remember that one washout of the bank will cost much more than the increased cost of a larger culvert, even disregarding the delay to traffic that would result, and the possible loss of human life.

For practical reasons, no covered culvert should be built less than 2 feet wide by 3 feet high, because of the danger of clogging up and the difficulty of effecting the consequent clearing out.

COVERS OF BOX CULVERTS

37. Formula for Thickness.—The thickness of cover required for a box culvert is governed by: (1) the amount and character of the load it must bear; (2) the span, or width of opening; (3) the transverse strength of the material of which the cover is composed.

The required thickness may be determined by the formula

$$t = \sqrt{\frac{3\,W l^2}{4 j_1 R}}$$

in which t = thickness of cover, in inches;

W = weight, in pounds per square foot, of uniformly distributed load (or the equivalent distributed load if the loads are concentrated);

l = effective span, in feet, which may ordinarily be taken as $1\frac{1}{4}$ times the clear opening;

R = modulus of rupture of the material used, in pounds per square inch;

j_1 = reciprocal of factor of safety ($j_1 R$ = safe intensity of stress in bending, in pounds per square inch).

38. Loads.—The load to be carried is the weight of the bank above the cover and the heaviest load (usually a locomotive) that is likely to be on the bank, with the proper allowance for the effect of impact and imperfect distribution of the load. The weight of materials of which such banks are usually composed varies from 75 pounds per cubic foot for dry loam to 140 pounds per cubic foot for quartz sand thoroughly wet, probably seldom reaching either of these extremes for the average of the whole height of embankment; it may generally be taken with safety as 125 pounds per cubic foot.

The weight on the drivers of the heaviest locomotive in common use may be taken as 50,000 pounds per axle, with axles 5 feet on center, or an average of 10,000 pounds per foot of track. Though the rails and ties do not make a perfect distribution of this load, a few feet depth of fill will probably distribute the load over a width of at least 10 feet, making 1,000 pounds per square foot, and absorb the effect of impact. This load may be safely used for the design of culvert covers if the height of bank is 10 feet or more. With a lower bank, the distribution of the live load will be less perfect, and the effect of impact will be greater; so that it will be advisable to design the cover as if the actual height of the embankment were 10 feet, with the distribution of the live load as before. This will give for W, in Art. **37,** a value of 2,250 pounds per square foot, being the sum of the live load (1,000 pounds per square foot) and the weight (125×10) of the assumed embankment. It should be understood that this height of embankment is assumed merely for the purposes of calculation.

Though it is not explicitly acknowledged by engineers, it is the generally accepted opinion, which accords with common practice in dimensioning, that the arching over of the materials of the bank effects a considerable reduction in the stress developed in the cover, and that a cover calculated for the full load of a 10-foot bank and a live load as just given will be ample for any depth of fill, unless the bank is composed of sand or soft clay. Sand, when either dry or thoroughly wet, has very little tendency to arch over; when moderately damp, it will do so. Accordingly, a load of 2,250 pounds per square foot may be used, except when the bank is composed of sand or soft clay.

39. Working Stresses.—The modulus of rupture of different stones used for culvert work varies greatly, even for apparently perfect and uniform stone of the same class. The variation, in pounds per square inch, is from 900 to 2,700 for granite, with an average of 1,800; 550 to 2,400 for sandstone, with an average of 1,250; 200 to 2,500 for limestone, with an average of 1,500; and 400 to 4,500 for bluestone flagging, with an average of 2,700.

It is not safe to use so large a modulus of rupture as the average, unless a larger factor of safety is also used. With proper care in culling out all stone showing defects, it is safe to use, for working stresses ($= j_1 R$, or the modulus of rupture divided by the factor of safety), 450 pounds per square inch for granite or bluestone flagging, and 250 pounds per square inch for sandstone or limestone of good quality. By substituting the values given, namely:

$W = 2{,}250$ (unless the fill is of sand or soft clay, in which case 125 pounds per square foot should be added for each foot of depth of fill over 10 feet);

$$j_1 R = \begin{cases} 450 \text{ for granite or bluestone,} \\ 250 \text{ for good quality of sandstone or limestone;} \end{cases}$$

we have, for granite or bluestone,

$$t = \sqrt{\frac{3 \times 2{,}250\, l^2}{4 \times 450}} = 1.94\, l \qquad \textbf{(1)}$$

and for sandstone or limestone,

$$t = \sqrt{\frac{3 \times 2{,}250\, l^2}{4 \times 250}} = 2.60\, l \qquad \textbf{(2)}$$

40. It is customary to make the thickness of the cover stone for box culverts equal to one-fourth the width of opening (or 3 inches per foot) for granite, and one-third (or 4 inches per foot) for sandstone and limestone; this makes an allowance for the spaces between cover stones. Bluestone makes a very good cover for small openings, but can seldom be obtained in good condition large and thick enough for large culverts.

41. Materials for Covers.—Cover stones should have a good level bearing on each wall of not less than their depth, with a minimum of 12 inches. Covers are often laid with open joints between the stones, it being claimed that this facilitates the drainage of the fill above. This construction, however, is objectionable, because it tends to cause clogging of the culvert.

42. Hard-pine timber used for culvert covers is subject to rapid decay, and at least 3 inches extra thickness should be allowed for it, with an intensity of stress of 800 pounds per square inch for the remainder of the timber.

43. Plain concrete is not generally used for the coverings of box culverts, because, for a given size of opening, an arch is stronger and not more expensive. Plain concrete may, however, be used with working stresses of 150 pounds per square inch for a 1 : 2 : 4 concrete, or 75 pounds per square inch for a 1 : 3 : 6 concrete (a good brand of Portland cement with proper sand and broken stone or gravel being used).

For good concrete reinforced with rods of mild steel, well designed and properly mixed and placed, the working stresses, in pounds per square inch, may be 12,500 to 15,000 pounds tension on steel rods; 625 pounds compression on concrete for a 1 : 2 : 4 mixture; and 500 pounds compression on concrete for a 1 : 3 : 6 mixture.

SIDE WALLS OF BOX CULVERTS

44. The thickness of the side walls of box culverts should, theoretically, be determined by the same rules that are applicable to retaining walls, but the effect of the bank over the top of the culvert must be kept in mind. The resistance to the horizontal thrust from the surcharging bank, given by that portion of the bank which is on the other side of the culvert and lies above the cover, greatly reduces the thickness of the wall required from what would be needed at the bottom if the wall were carried up to the top of the fill. Experience indicates that the thickness of the side walls of culverts, with nothing to serve as bracing to hold them apart either at top or bottom, should not be less at the top than one-half the height of the culvert, nor at the bottom less than two-thirds that height.

In timber culverts, the walls not being designed to resist by their weight the thrust of the bank, as must be done with stone culverts, it is very important that the walls should be securely drift-bolted together, as well as to the pavement (or sills) and the roof. With walls properly so drift-bolted, the side walls may be made of a uniform thickness of one-fourth the height of the culvert, with 2 or 3 inches additional for decay.

45. In stone culverts, all the stone in the side walls should be sound, roughly squared, and laid so as to be well bonded together. Not less than one-half of the top course, and one-quarter of the lower courses, should pass entirely through the walls; these through stones should be of good size and be selected especially for the duty of binding the wall together.

In double or multiple culverts, the thickness of the partition wall should not be less than one-half the height, and must provide ample bearing for the ends of cover stones. All the top course and not less than one-half the bulk of the lower courses of these partition walls should be of through stones; generally, all courses may be through stones.

DIMENSIONS OF ARCH CULVERTS

46. Definitions.—The parts of an arch that are now to be defined are illustrated in Fig. 2.

The **span** $a b'$ is the perpendicular distance between the faces of the walls A and B supporting the arch.

47. The **springs**, or **spring lines**, are the lines of junction of the arch with the supporting walls; these lines run longitudinally with the arch, and are represented in the cross-section by the points a and b'.

48. The **crown** de is the central and highest part of the arch.

49. The **rise** cd is the vertical height from the level of the spring lines to the lower face of the crown; it is the middle ordinate of the arc $a d b'$ forming the under face of the arch.

50. The **soffit** is the lower concave surface of the arch, and is represented in the figure by the line $a d b'$. The soffit is also called the **intrados**; the latter term is often applied to the line of intersection, as $a d b'$, of the soffit with a vertical plane normal to the axis of the arch.

51. The **extrados** is the outer convex surface of the arch, or the line of intersection of this surface with a vertical plane normal to the axis of the arch (the line $s e t'$, or the surface of which this line is the projection). When the arch stones, or **voussoirs,** are distinct from the spandrel filling, as is usual with stone arches, the joint $u e v$ between the arch stones and the spandrels is more properly called the extrados. Even though the structure may be monolithic, the same imaginary surface or line is also so designated for mathematical purposes. The term **back** is also applied to the extrados surface.

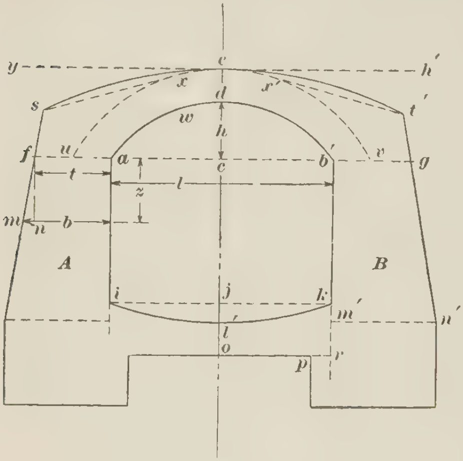


Fig. 2

52. The **haunches** are the parts of the arch lying between the crown and the springs. There is no definite division line between the crown and the haunch; the latter term may properly be applied to the part $a f s x w$, or may be used for the whole area $a f s e d$ to the center of the arch.

53. The **key** is the central highest part of the arch. When the arch is composed of separate stones, the stone at the key is called the **keystone.**

54. The term **spandrel** is applied to the space over the haunches of the arch and below the level of the top of the crown. In Fig. 2, $u e y$ and $v e h'$ are the spandrels. The material occupying this space, whether of masonry or earth, is called the **spandrel filling.** The parts $u e s f$ and $v e t' g$ of the spandrel filling that are composed of masonry are called **backing.**

55. The **invert** is the *inverted arch* $i\,l'k$ at the bottom of the culvert, and is subjected to stresses similar to those in the main arch, but in the opposite direction. The difference $j\,l'$ in level between the spring lines and the crown of the invert is called the **dip** or **dish** of the invert, and corresponds to the rise of the upper arch.

56. Radius of Intrados.—Arches for culverts are usually made in the form of a circular arc. If the arc is semicircular, the arch is called a **semicircular** arch; if the arc is less than a semicircle, the arch is a **segmental** arch. The semicircular form is more common, although, theoretically, the segmental form is the better form.

The following formula gives the radius R of the intrados $a\,d\,b'$, Fig. 2:

$$R = \frac{h}{2} + \frac{l^2}{8\,h} \qquad (1)$$

in which h = rise $c\,d$;

l = span $a\,b'$

If the arch is semicircular,

$$R = h = \frac{l}{2} \qquad (2)$$

57. Depth of Key.—The factors governing the thickness of an arch at the key and in the haunches are: (1) the amount and character of the load the arch must bear, which is the same as though the culvert were of box form (see Art. **38**); (2) the span; (3) the rise; and (4) the compressive strength of the material of which the arch is made. These are the factors governing the thickness of an arch; but in the common formula for determining the depth of key, the load and the strength of material do not directly appear; however, they were taken into account in selecting the constants contained in the formula. This formula, which is mainly empirical, was first suggested by John C. Trautwine in his well-known Pocketbook, and is now very extensively used; it is as follows:

$$K = \frac{\sqrt{R + \frac{l}{2}}}{4} + .2 \qquad (1)$$

Here R and l have the same significance as in Art. **56,** both being in feet; and K is the depth of key, in feet. If the arch is semicircular, $R = \frac{l}{2}$, and the formula becomes

$$K = \frac{\sqrt{l}}{4} + .2 \qquad (2)$$

These formulas apply to first-class cut-stone arches. For second-class stonework, the results should be multiplied by $\frac{9}{8}$, and for rubble, brick, or concrete, by $\frac{4}{3}$.

Example 1.—The span of an arch is 8 feet, and the rise, 1 foot 9 inches; to determine the radius.

Solution.—Here, $h = 1.75$ and $l = 8$. Substituting these values in formula **1,** Art. **56,**

$$R = \frac{1.75}{2} + \frac{8^2}{8 \times 1.75} = 5.45 \text{ ft.} \quad \text{Ans.}$$

Example 2.—The span of an arch is 6 feet, and the rise, 1 foot 6 inches; what depth of key should be used for second-class stonework?

Solution.—To apply formula **1,** Art. **57,** we have $l = 6$ and $R = \frac{1.5}{2} + \frac{6^2}{8 \times 1.5} = 3.75$. Substituting in the formula and multiplying by $\frac{9}{8}$ for second-class work,

$$K = \frac{9}{8}\left(\frac{\sqrt{3.75 + \frac{6}{2}}}{4} + .2\right) = .96 \text{ ft., or } 11\tfrac{1}{2} \text{ in.} \quad \text{Ans.}$$

58. Thickness of Walls.—The thickness of the abutments, or side walls, of arch culverts required to resist the thrust of the arch may be found by the following formulas (see Fig. 2):

$$t = \frac{R}{5} + \frac{h}{10} + 2 \qquad (1)$$

$$b = t + \frac{l\,z}{24\,h} \qquad (2)$$

in which R, h, and l have the same significance as in Art. **56;** t represents the thickness, in feet, of the abutment at the level of the spring lines; and b is the thickness, in feet, of the abutment at a distance of z feet below the spring line.

These formulas give the thickness of abutment necessary to resist the thrust of the arch. But if the thickness so obtained for the bottom of the abutment is less than two-thirds the height of the wall to the spring line, it will be insufficient to resist the thrust of the bank, and the thickness at the bottom should be increased to two-thirds of the height, while the top should be thickened by the same amount.

The required thickness of abutments is independent of the quality of masonry, except that, if voids exist, allowance must be made for them.

If the arch is semicircular, $R = h = \frac{l}{2}$, and formulas **1** and **2** become, respectively,

$$t = \frac{3l}{20} + 2 \qquad (3)$$

$$b = t + \frac{z}{12} \qquad (4)$$

59. Batter.—The inner surface of each wall is made vertical; the back of the wall, inclined (see Fig. 2). The batter of the back is expressed as follows: In the vertical distance fn, or z, the back deviates from the vertical by the amount mn, or $b - t$; therefore, the horizontal deviation I, in feet per foot vertical, is given by the equation

$$I = \frac{b - t}{z}$$

which, by inserting the value of b from formula **2** of Art. **58**, becomes

$$I = \frac{l}{24h} \qquad (1)$$

For the batter i, in inches horizontal per foot vertical, we have

$$i = 12I = \frac{l}{2h} \qquad (2)$$

Example 1.—What thickness at top and bottom is required for the wall of an arch of 6 feet span and 1 foot 6 inches rise, the walls being 4 feet 6 inches high to the spring line?

Solution.—Here, R (as found in example 2, Art. **57**) = 3.75 h = 1.5, l = 6, and z = 4.5. Then, using formulas **1** and **2**, Art. **58**,

$$t = \frac{3.75}{5} + \frac{1.5}{10} + 2 = 2.9 \text{ ft.}$$

and

$$b = 2.9 + \frac{4.5 \times 6}{24 \times 1.5} = 3.65 \text{ ft. Ans.}$$

Example 2.—What batter should be given to the back of abutments for arches of which the ratios of span to rise are: (*a*) 2? (*b*) 4? (*c*) 8?

Solution.—(*a*) Since the ratio of the span to the rise is 2, $\frac{l}{h} = 2$. Substituting in formula **2**, $i = \frac{2}{2} = 1$ in. horizontal to 1 ft. vertical. Ans.

(*b*) In this case, $\frac{l}{h} = 4$. Substituting in formula **2**, $i = \frac{4}{2} = 2$ in. horizontal to 1 ft. vertical. Ans.

(*c*) Similarly, $i = \frac{8}{2} = 4$ in. horizontal to 1 ft. vertical. Ans.

60. Thickness of the Haunches.—The thickness of the wall at the level of the spring line having been determined, as well as the batter of the back, this batter should be continued upwards to a point s, Fig. 2, midway between the level of the springs and that of the extrados at the crown. From this point, the top of the haunches may be made a plain surface tangent to an extrados $u\,e\,v$ made concentric with the intrados from the crown to its intersection x with the said plain surface, the top taking the form indicated by the dotted line $s\,x\,e\,x'\,t'$. It is customary (especially if concrete is used) for such small arches as are used in culverts to make the top a cylindrical surface, as indicated by the full line $s\,e\,t'$; this takes but little extra material with so small an arch, and makes a better and more easily defined surface.

If the structure is to be of stone masonry, the part outside the line $u\,e\,v$ may preferably be laid in horizontal courses, while the part inside this line should have the joints radial. The top surface should be smoothed off and made sufficiently water-tight, so that it will drain off. A troweled surface of wet concrete makes a good finish.

EXAMPLES FOR PRACTICE

1. What depth of key is required for a concrete arch culvert of 12-foot span, with a rise of 2 inches per foot of span?

Ans. 1 ft. 8 in.

2. What is the radius of the intrados, and what the depth of key, for a circular arch culvert of cut stone, the span and rise being 10 and 3½ feet, respectively?

Ans. $\begin{cases}\text{Radius, 5 ft. } 3\tfrac{7}{8} \text{ in.}\\ \text{Key, 1 ft.}\end{cases}$

3. What depth of key is required for a semicircular brick arch culvert having an 8-foot span? Ans. 1 ft. $2\frac{1}{2}$ in.

4. In a concrete arch culvert of 12 feet span and 12 feet clear height at the crown, and having a rise of 2 inches per foot of span, what are the dimensions of the cross-section of the abutments?

Ans. Height, 10 ft.
Thickness at spring, 4 ft. $2\frac{3}{8}$ in.
Thickness at base, 6 ft. $8\frac{3}{8}$ in.

5. What is the thickness of the walls at the spring line and at the base for a circular arch culvert of cut stone, the span and rise being, respectively, 10 and $3\frac{1}{2}$ feet, and the height of wall 6 feet?

Ans. At the spring, 3 ft. 5 in.
At the base, 4 ft. $1\frac{1}{2}$ in.

PAVEMENT

61. A great many culvert failures are due to the undermining of the walls by the scour of the water rushing through. To avoid this danger, a pavement should be provided. Too little care in paving is common because its importance is not sufficiently realized; frequently, wedge-shaped or pyramidal stones are used, with the natural result that the pressure at the sides (or the upward pressure from the bottom if the soil is soft) dislodges them, destroying the pavement and eventually causing the ruin of the whole structure. Projections or irregularities in the upper surface may cause the lodging of drift and the clogging of the waterway, or give an opportunity for the drift to strike injurious blows.

The pavement should be made impervious, for, if water is allowed to pass through it, it is likely to find a channel underneath, either through voids already existing, or made by the softening or settling of the soil, and the scour proceeds as though no pavement had been laid. This danger is all the worse because it is out of sight and perhaps not realized. A casual inspection may reveal the trouble if a part of the pavement has been washed away, but underground watercourses may grow undiscovered.

A culvert may last a long time with a poor pavement or with no pavement, but the greater safety afforded by a good pavement is well worth its cost.

62. Cross-Walls.—As the velocity, and therefore the scouring action, of water is greatly diminished by a barrier that changes the direction of flow, **cross-walls**, also called **cross-sills**, are built at intervals under the culvert, extending from one side wall to the other.. A cross-wall may frequently cause a cavity to fill up when otherwise a dangerous underground channel might form. Cross-walls are especially desirable if the culvert is long or has a steep grade. These walls should be made of large stone, with the top level with the top of the pavement, and their base at least 1 foot below the pavement.

63. Aprons and Apron Walls.—The danger of undermining is greatest at the ends of the culvert: at the upper end, before the water has been confined; and at the lower end, where the soil may be easily carried away by the stream. To prevent the scour from starting at these two points, the pavement is carried beyond the ends of the culvert at least as far as the wing walls extend, and farther if necessary for protection. This extension of the pavement is known as the **apron.** At the end of the apron, a cross-wall, called an **apron wall** or a **curb wall,** is built. If the apron is carried beyond the ends of the wing walls, an apron wall should be laid not only at the end of the apron, but also under the ends of the wings. If there is much fall to the ground surface below the mouth of the culvert, or if water stands about it, riprapping may be required for the safety of the culvert, as well as for the protection of the bank.

64. Stone Pavement.—Stone pavements should be not less than 12 inches deep. The stone should be sound, with nearly vertical sides, and nearly uniform in depth and transverse dimensions. Large stones are desirable for paving, but the allowable depth should not be reduced when they are used. The pavement should always be laid in mortar, or else it should be grouted after the stone is placed. This is more important for the pavement than for the walls and cover, especially when the foundation is soft or of a fine material.

Engineers differ as to the best design for pavements for stone box culverts: some prefer to excavate to a uniform depth and extend the pavement under the side walls; while others carry the walls deeper than the pavement, and pave only between the walls. The former plan makes the pavement an integral part of the structure, and reduces the danger of cracks being formed at the face of the walls where the walls join the pavement; when such cracks are formed, water runs through them and undermines the structure. The other design reduces the danger from frost, and frequently affords a firmer foundation.

65. Concrete Pavement.—In concrete culverts, especially in arch culverts, the pavement is usually made of the same material and with an invert; this pavement is readily made, and combines the advantages of the two plans described in the last article.

The form of the invert offers as much opportunity for variety as the roof arch itself—from a perfectly flat surface to a semicircular form. When the invert is made in the form of a circular arc, its thickness at the center may be determined by the formula used for an upright arch, by substituting the dishing of the invert for the rise h of the arch, and multiplying the result by from $\frac{1}{3}$, for a very firm foundation, to 1, for a soft foundation, according to the nature of the foundation. For an average foundation, multiply by $\frac{3}{4}$.

66. Considering the invert as a pavement, it seems wise, with a material so easily shaped as is concrete, so to lower the center below the sides that, when there is a very small volume of water, its course may be directed through the culvert instead of being collected in pools too shallow to run off, as might be the case if the pavement were flat. On the other hand, with a semicircular invert, or one with a large amount of dishing in proportion to the span, the waterway is reduced too much in its lower and first affected portion. A dishing of about 1 inch per foot of span seems to satisfy all requirements. The invert may consist of two straight slopes meeting at an angle at the center, as shown in Fig. 17, or of two straight lines connected by a curve at the center, as shown in Fig. 16. In the former case, the slope of the pavement at the line of the face of the abutments is 2 inches per foot, while in the latter case it is at the rate of 4 inches per foot.

67. Cross-walls should be built under the invert of a concrete culvert, as indicated in Art. **62,** the same as if stone masonry were used. If the grade is steep, the foundations may be prepared in steps to give a succession of level bearings for the concrete, and the opening in the culvert may be made on an even grade; or the opening also may be stepped, as suggested in a subsequent article. Where practicable, the former method is preferable.

68. Timber Pavement.—The floor of timber culverts may be made by leveling off and compacting the soil and laying transversely on it timbers 6 or 8 inches thick. These timbers should be laid close; they should be made somewhat longer than the distance out to out between the walls of the culvert, and the first timbers of the walls should be drift-bolted to about every third floor timber.

An alternative arrangement that is somewhat less expensive but less durable is to place mud-sills at the toe of the slope at each side of the bank, and at intervals of about 6 feet—all with the tops on an even slope. These sills should be well and firmly bedded in the soil, and a smooth and firm surface made between them on the plane of the tops of the sills; on these sills is laid a floor of longitudinal plank 2 or 3 inches thick having the same width as the culvert, and being well spiked to the sills. The wall timbers are set outside this floor and drift-bolted to each sill. It is advisable so to space the sills that one will come under each joint in the first wall timbers, and each timber be drift-bolted to it.

CULVERTS WITH A GRADE

69. Culverts with a heavy grade may often be built with steps a few inches in height, having their tops slightly inclined in the direction of the grade. When this is done, the step in the cover should be made a little nearer (say about three times the amount of drop) to the lower end of the culvert than the step in the pavement; otherwise, the capacity of the culvert is reduced, and the danger of clogging is correspondingly increased. If the step in the cover can be relieved by using a beveled stone, this will reduce the danger of catching drift. A succession of steps from 6 to 10 inches in height is preferable to one large step of the same total drop. If the natural grade of a stone culvert exceeds about 10 per cent., it is preferable to use steps in this way, so that the bed of each stone may be laid on the bottom with comparatively little inclination, and be therefore in no danger of sliding. Wherever such steps are

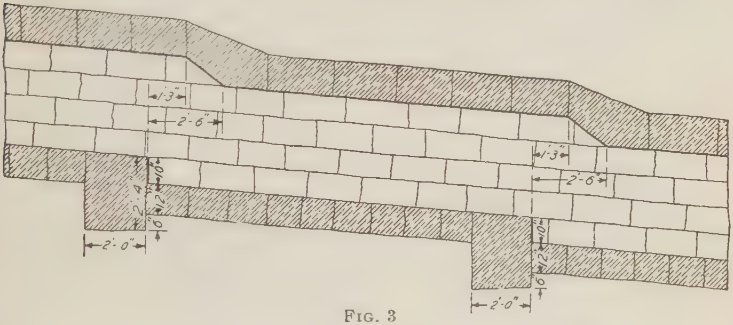


Fig. 3

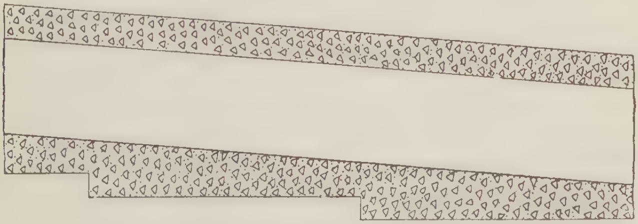


Fig. 4

introduced, cross-walls should be used; these walls should be at least 2 feet deep and extend 6 inches below the bottom of pavement at the foot of the step. Fig. 3 shows a longitudinal section of a stone culvert built on a grade.

If the culvert is of concrete, the same method may be adopted, but it is preferable, where possible, to leave the opening of the culvert on a uniform grade and step off the foundations to give a succession of level bearings, as shown in Fig. 4.

70. If the foundation of a culvert is on rock in which the natural grade exceeds 10 per cent., advantage may be taken of this natural pavement; but that part of the rock that supports the walls should be dressed off to give approximately a level bed for each stone resting on it, and the walls should be laid in nearly horizontal courses, the cover being stepped down at intervals as may be required.

WING WALLS AND PARAPETS

71. Object of Wing Walls.—As the covered part of a culvert is rarely extended as far as the toe of slope of the bank, **wing walls,** or **wings,** are built at each end to prevent the fill from running into or across the mouth of the culvert. The wings serve also to direct the course of the water into or away from the culvert so as to prevent its getting behind the main walls.

72. Kinds of Wing Walls.—Wing walls are divided into *straight wings*, *flaring wings*, and *transverse wings*. **Straight wings** are continuations of the culvert walls in the same direction as the body of the culvert; **flaring wings** are turned back at an angle with the culvert; **transverse wings** are at right angles with the culvert, or are parallel with the bank if the culvert is skewed.

73. Straight wings are generally used for stone box and for timber culverts, unless the direction of the watercourse demands another form of wing. Straight wings are the most economical, and are ordinarily stepped down according to the slope of the bank (see Fig. 5).

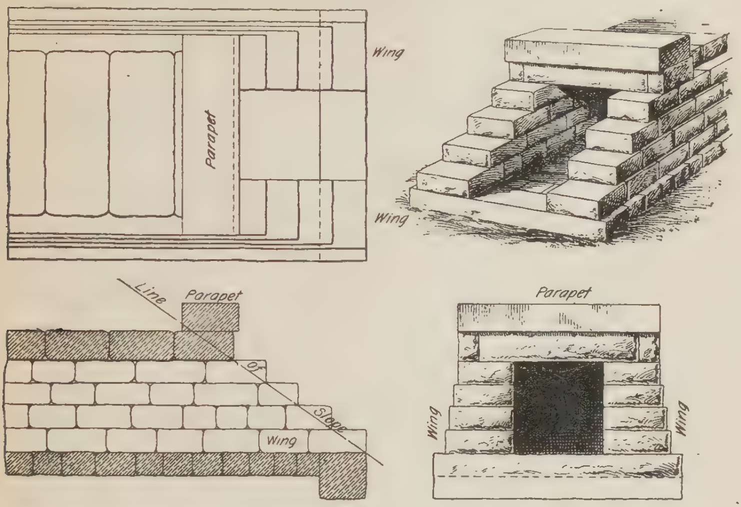


FIG. 5

74. Flaring wings naturally accompany arch culverts, and also concrete culverts, even if the cover is flat. The enlarged inlet and outlet they form aid in increasing the capacity of the culvert, by easily directing the water into it,

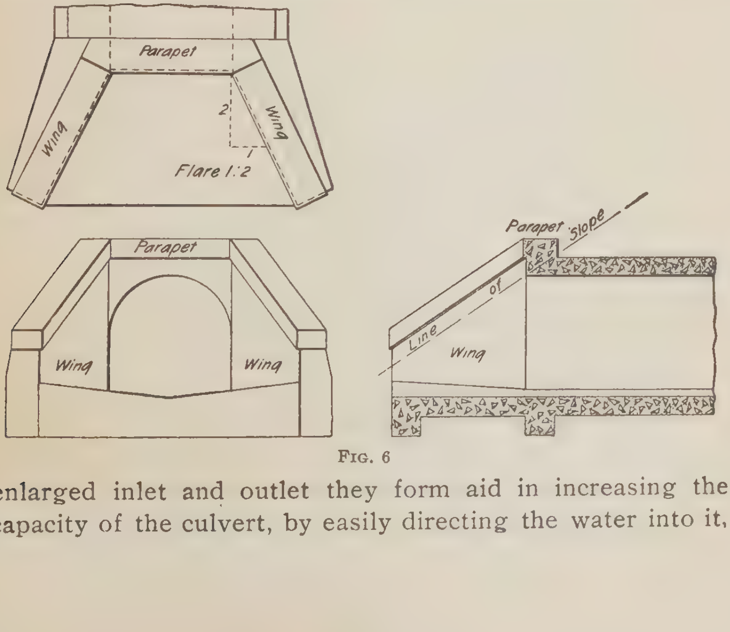


FIG. 6

and allowing the discharge to spread and so decrease in depth before leaving the line of the bank and the protection given by the pavement (see Fig. 6). The amount of **flare, or splay,** as the deviation of the wings from the direction of the culvert walls is called, need not be very great; a flare of 1 : 2 (see Fig. 6) is generally sufficient. With a large flare, the length of wing and the quantity of masonry are increased with no corresponding gain in efficiency.

As with straight wings, flaring wings may be stepped down or sloped to correspond with the bank they restrain.

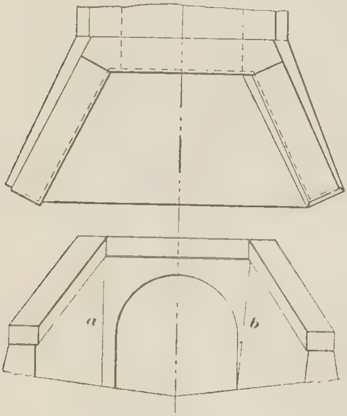


Fig. 7

In joining the wings with the abutments or end walls, there should be no offset made in the face of the masonry, for such jogs will cause eddies, and so retard the water in its flow, besides adding materially to the expense; to this must be added that, at the upper end, they will facilitate the accumulation of drift. The junction should be as shown in Fig. 6, not as shown at *a* or at *b*, Fig. 7. For this reason, the faces of wing walls should not be battered, unless either the end wall or the abutment inside the culvert is also battered. A vertical face of the side wall is sometimes continued beyond the ends of the cover, and is intersected with a battered face of the wing outside the culvert; but this adds to the expense of the forms without any corresponding advantage, the supposed esthetic superiority being the only excellence claimed.

75. Transverse wings are sometimes used for either stone or concrete culverts, whether of box or arch form, and are the common form for pipe culverts. They are carried to a uniform height for their whole length, and the slope is allowed to run around in front, the length being determined by this slope. Because of the ends of the wings being covered with the slope, there is less danger of their being undermined than with straight or flaring wings. Fig. 8 illustrates the transverse wings.

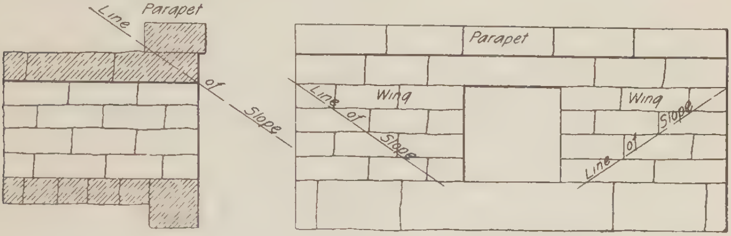


Fig. 8

76. Parapets.—The length of stone or concrete culverts is generally reduced by building **parapet walls** across the line of culvert at the end of the cover, extending the full width of the culvert and side walls, or the whole length of the wings if transverse wings are used. For appearances, the top course of the parapet is generally made to project from 2 to 4 inches beyond the face of the cover and lower courses (if any) of the parapet (see Figs. 5, 6, and 8).

FOUNDATIONS

77. The principles governing the foundations of culverts are the same as for other foundations; but, on account of the large areas covered by culverts in proportion to their expense, it is seldom that the same care is warranted in preparing the foundation that would naturally be taken for a bridge pier or abutment. When the soil is gravel or a moderately firm earth, it is customary to excavate only as required for the pavement and the walls; but, if the bottom is swampy, a good foundation may be formed by timbers or logs laid longitudinally and transversely. *Timber should not be so used unless the ground will always remain damp*, for otherwise the timber will rot, and the foundation will be worse than if no timber had been used. If the soil is soft and compressible, but unsuitable for the use of timber, it may be excavated to a greater depth, and made more firm and compact by tamping sand, gravel, or broken stone into the softer material.

78. If the culvert rests on rock, the pavement is omitted, but all rotten stone and loose scale should be removed, and the surface prepared so as to give a good level bed for the masonry. If the soil is partly rock and partly softer material, great care must be taken to avoid settlement and a break in the culvert at the point of change, and also to make a safe joining of the paving to the rock.

79. When the foundation is soft or varies greatly in character, there is danger of transverse breaking or distortion of the culvert by the uneven settlement. The writer has seen a culvert that was built on a uniform grade under a high bank across soft ground which had settled so much that one could not see from end to end, and water was standing nearly to the roof at the middle when both ends were dry. Where settlement is expected, the middle of the culvert may be laid higher than the ends; but no empirical rule can be given for the amount of settlement: judgment founded on individual experience is the only satisfactory guide.

LENGTHS

80. The length of a culvert is governed by the height of the bank and the width of the roadbed, and by the style of end to be used. Generally, it is so chosen that the plane of slope of the bank shall come just to the top of the culvert at the end of the cover.

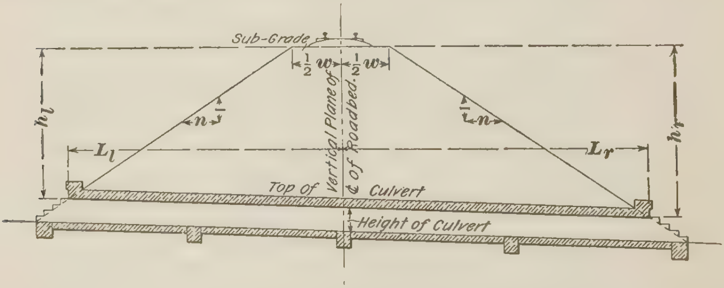


FIG. 9

As the two ends of a culvert are seldom at the same level, the length of each side must be determined separately. The following formulas may be used (see Fig. 9):

$$L_l = \frac{w}{2} + n\,h_l \qquad (1)$$

$$L_r = \frac{w}{2} + n\,h_r \qquad (2)$$

in which

L_l and L_r = horizontal lengths from center line of roadbed to end of cover, on the left and right sides, respectively, measured at right angles to the roadbed;

w = width of roadbed at subgrade;

h_l and h_r = difference in elevation between subgrade and top of culvert at the left and right sides, respectively;

n = slope ratio (horizontal distance divided by corresponding vertical distance) of side of bank (commonly $1\frac{1}{2}$, but may be more if the bank is composed of sand or clay, or less if of large loose rock).

If it is desired that the length to the ends of the wings should just reach the toe of the slope, L may be considered as the length to, and h the height of the bank above, this point, instead of referring to the end of the cover. If the culvert is on a skew with the bank, the center line and subgrade elevation opposite the end of the culvert must be used instead of that over the axis of culvert. If the parapet and wings are to be built normal to a skew culvert, allowance must be made for the skew in the width of the structure. When the culvert has a heavy grade, the wings should be lengthened at the lower end, and may be shortened at the upper end.

Example.—What is the horizontal length of cover from the center line of an 18-foot roadbed for a culvert 4 feet high, the elevation of subgrade being 117.2 and that of the bottom of culvert at the end being 94.7, the slope of bank being 1.5 : 1?

Solution.—Here, $w = 18$, $n = 1.5$, and $h_r = 117.2 - 94.7 - 4 = 18.5$. Substituting in formula **2**,

$$L_r = \tfrac{18}{2} + 1.5 \times 18.5 = 36\tfrac{3}{4} \text{ ft.} \quad \text{Ans.}$$

DETAILS AND ILLUSTRATIONS OF CULVERT CONSTRUCTION

PIPE CULVERTS

81. Pipe culverts, whether of terra cotta or of cast iron, are sometimes laid with no other preparation than simply hollowing out the soil to receive the pipe, or excavating a trench and filling it with sand up to the middle of the pipe. If the soil is firm, the former method may prove satisfactory when proper care is taken to tamp well the filling required to replace the excess of material that is usually removed. Sand packed around the pipe makes an excellent bed if undisturbed, but even a slight leak in the pipe may wash away the sand. The best practice is to lay the pipe in a bed of concrete extending up to the middle of the pipe.

82. Whether the pipe is set in a concrete bed or rests directly on the soil, all joints should be calked or laid in cement. The pipe should always be laid with the bell end upstream. If practicable, there should be sufficient grade in pipe culverts, especially at the lower end, to prevent settlement from forming pockets in which water may collect and freeze. When the ground is soft and only a slight fall is possible for the whole length of culvert, it is wise to raise the central part of the culvert even above the inlet, as the

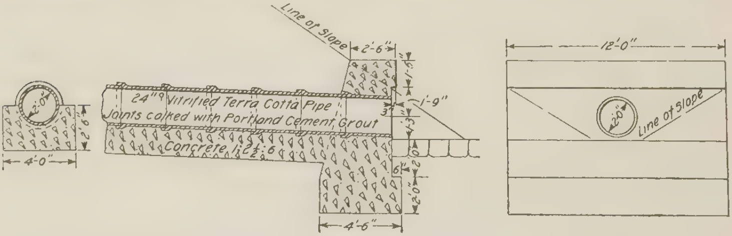


FIG. 10

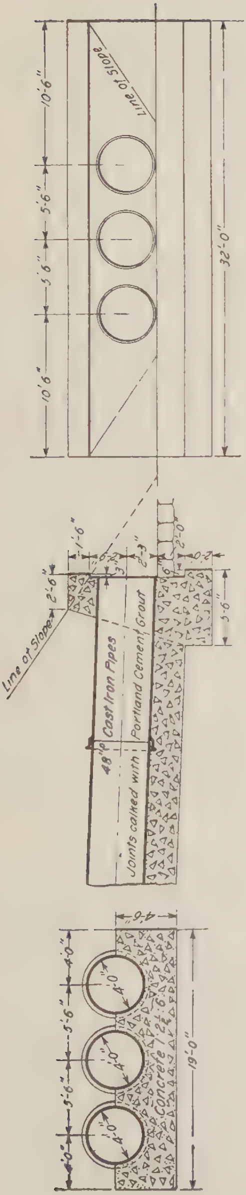


FIG. 11

impounding of a small quantity of water at the upper end of the culvert is less serious than to allow water to collect within the pipe. The ends of the pipes should be protected by head-walls of concrete or stone masonry, which should be long and high enough to prevent obstruction of the mouth of the pipe by the wash of the bank. The head-walls should be carried to such a depth as to avoid the dangers from frost and undermining. With pipe culverts, liberality in the length of the culvert may effect a saving in the cost of head-walls equal to many times the extra cost of pipe. Unless the flow through a pipe culvert is expected to be small, a pavement should be laid below the outlet, or riprap should be used.

Fig. 10 shows a single terra-cotta pipe culvert, with 24-inch pipe, and Fig. 11 shows a triple cast-iron pipe culvert with 48-inch pipe.

83. Pipe culverts are used largely to replace decayed timber culverts. The timber flooring may be removed, and even the walls and roof may be partly cut away for the brief time required for the work, so that pipe of as large an inside diameter as the smaller dimension of the timber culvert may readily be substituted without interfering with the bank. The space outside of the pipe should be packed earth and spalls. This method of building temporary culverts of timber and replacing them later with pipe is especially satisfactory, because during the life of the timber the bank will settle and the engineer can see whether his estimate for the size of the opening was correct. Timber culverts may be made larger than absolutely required, so as to allow pipe of the estimated capacity to be readily substituted when the time for renewal arrives.

TIMBER BOX CULVERTS

84. In timber culverts, as the walls are not designed to resist the thrust by their weight, the timbers should be thoroughly drift-bolted together (see Art. **44**). If the pavement is made of a succession of thick floor timbers laid close, the first timbers of the side walls should be bolted to the floor timbers about every 3 feet; or, if a thin floor is laid on sills, two bolts should be used at each end of each sill. The wall timbers should be butt-jointed and laid to break joints not less than every 4 feet, and have at least two drift bolts between adjacent joints. The different courses of wall timbers should be bolted together at intervals of about 5 feet, the bolts being long enough to reach two-thirds through the next lower timber.

85. The cover is made of transverse timbers laid close together, and of a thickness determined as stated in Arts. **37**, **38**, **39**, and **42**. Each of these cover timbers should be bolted to the top timber of the side walls, and every sixth or eighth cover timber should be notched down on the walls.

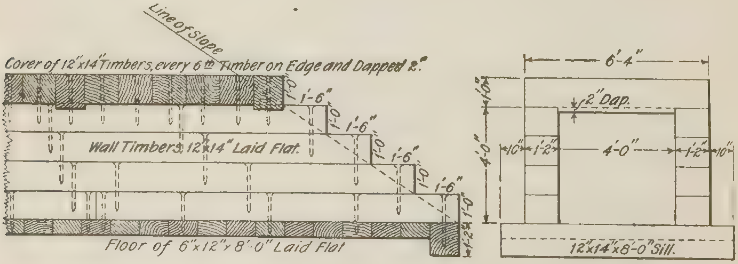

Line of Slope
Cover of 12"x14" Timbers, every 6th Timber on Edge and Dapped 2"
1'-0"
1'-6"
1'-0"
1'-6"
1'-0"
1'-6"
1'-0"
1'-6"
1'-0"
Wall Timbers, 12"x14" Laid Flat.
1'-2"
Floor of 6"x12"x8'-0" Laid Flat
6'-4"
1'-0"
2" Dap.
4'-0"
10"
1'-2"
4'-0"
1'-2"
10"
12"x14"x8'-0" Sill.

FIG. 12

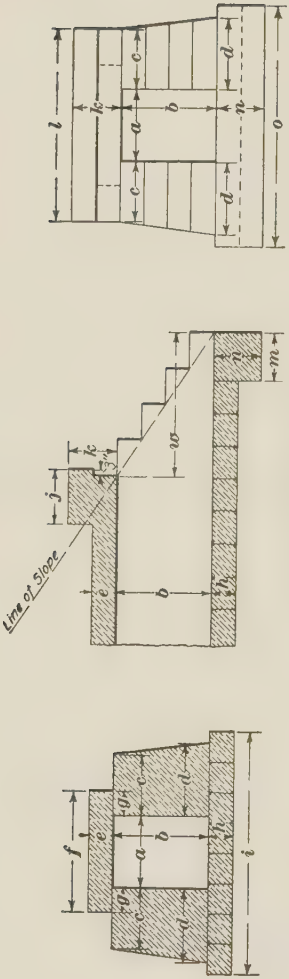
Timber culverts are commonly built without head-walls or parapets, but the ends of the side walls are stepped to serve as wings to retain the slope. Fig. 12 shows a typical timber box culvert 4 feet square.

STONE BOX CULVERTS

86. The cover and walls of a stone box culvert are proportioned as explained in Arts. **37** to **45**, inclusive. Their faces should be so smooth that there will be no danger of drift lodging or striking so hard as to dislodge a stone. The cover stones are generally laid with less care as to close joints than is customary for wall masonry, for their office is chiefly to act as beams to carry the bank above, and to some extent each stone does its work independently of the rest. In order, however, to make the cover tight, the openings between the stones should be filled with spalls laid in mortar.

87. The thickness of the walls at top and bottom, as determined in Art. **44**, is the minimum to be allowed, but it is not necessary that the back of wall should be dressed to a battered line; generally, the back will be rough, and the wall may be made as thick at the top as it must be made at the bottom. In estimating quantities for the contractor, however, a uniform slope on the back is used, as the quantity within this plane is what is absolutely required. No course or stone on the back of the wall should be allowed to overhang the one below.

88. A cross-section, a longitudinal section at the end, and an end view of a typical single box culvert of stone are shown in Fig. 13; and for a double box culvert, in Fig. 14. In the latter figure, the longitudinal section shows the middle wall; the side walls are the same as for a single box culvert. The dimensions of the different parts for these culverts are given in Table II, in which the dimensions not given for double culverts are the same as for single culverts. These dimensions are suitable for sandstone or limestone of good quality. The letters given in the table refer to Figs. 13 and 14.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Line of Slope
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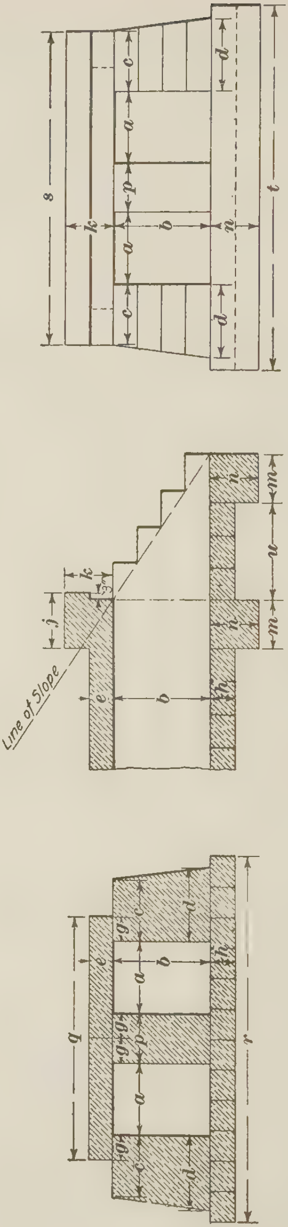
Fig. 13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Line of Slope
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3"
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p
r

Fig. 14

TABLE II

DIMENSIONS FOR STONE BOX CULVERTS

Size of Opening			Single Culverts																											Double Culverts											
			Thickness of Side Walls				Cover Stone						Pavement				Parapet						Apron Wall					Wings		Dimensions the Same as for Single Culverts, Except as Given Below											
Width *a*		Height *b*	At Top *c*		At Bottom *d*		Depth *e*		Length *f*		Bearing on Wall *g*		Depth *h*		Width *i*		Width *j*		Height *k*		Length *l*		Width *m*	Depth *n*		Length *o*		Length *w*		Thickness of Middle Wall *p*		Width of Cover *q*		Width of Pavement *r*		Length of Parapet *s*		Length of Apron Wall *t*		Between Apron Walls *u*	
Feet	Inches	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches
2		3	2		2	6		9	4		1		1		8		1	9	1	6	6		2	2		8		4	6												
2	6	3	2		2	6		10	4	6	1		1		8	6	2		1	9	6	6	2	2		8	6	4	6												
3		3	2	3	2	9	1		5		1		1		9	6	2	3	2		7	6	2	2		9	6	4	6	2		10		14	6	12	6	14	6	2	6
3		4	2	6	3		1		5		1		1		10		2	3	2		8		2	2		10		6		2		10		15		13		15		4	
3		5	2	9	3	6	1		5		1		1	2	11		2	3	2		8	6	2	2	2	11		7	6	2	6	10	6	16	6	14		16	6	5	6
4		4	2	6	3		1	4	6	8	1	4	1		11		2	6	2	8	9		2	2		11		6		2	8	13	4	17	8	15	8	17	8	4	
4		5	2	9	3	6	1	4	6	8	1	4	1	2	12		2	6	2	8	9	6	2	2	2	12		7	6	2	8	13	4	18	8	16	2	18	8	5	6
4		6	3		4		1	4	6	8	1	4	1	4	13		2	6	2	8	10		2	2	4	13		9		3		13	8	20		17		20		7	

89. Another design for stone box culverts is shown in Fig. 15. This form is preferred by many engineers; but, in the majority of cases, the form shown in Fig. 13 seems to be preferable, for the reasons stated in Art. **64.** The dimensions for the two designs would be the same, except for the arrangement of the paving and the bottom part of the walls. Double culverts are not used for openings less than 3 ft. × 3 ft.

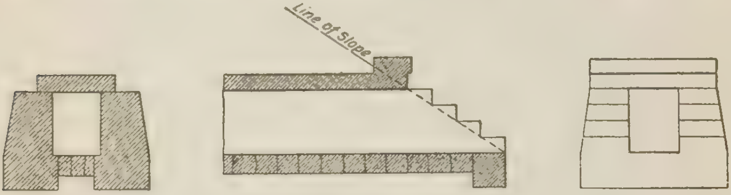


FIG. 15

STONE ARCH CULVERTS

90. Stone arches are rarely desirable for culverts with small openings, because this style of construction is too expensive for spans where ordinary stone covers are practicable. In places, however, where the appearance is important in determining the design, stone arches may be used, even though the span is small.

91. In the main, the design of stone arch culverts is similar to that of concrete arches; but, owing to the great cost of cutting the arch stones, a flatter arch would be suggested for stone than for concrete. The ratio of rise to span may be made from 1 : 3 to 1 : 6 for stone arches, according to the prices for material and work.

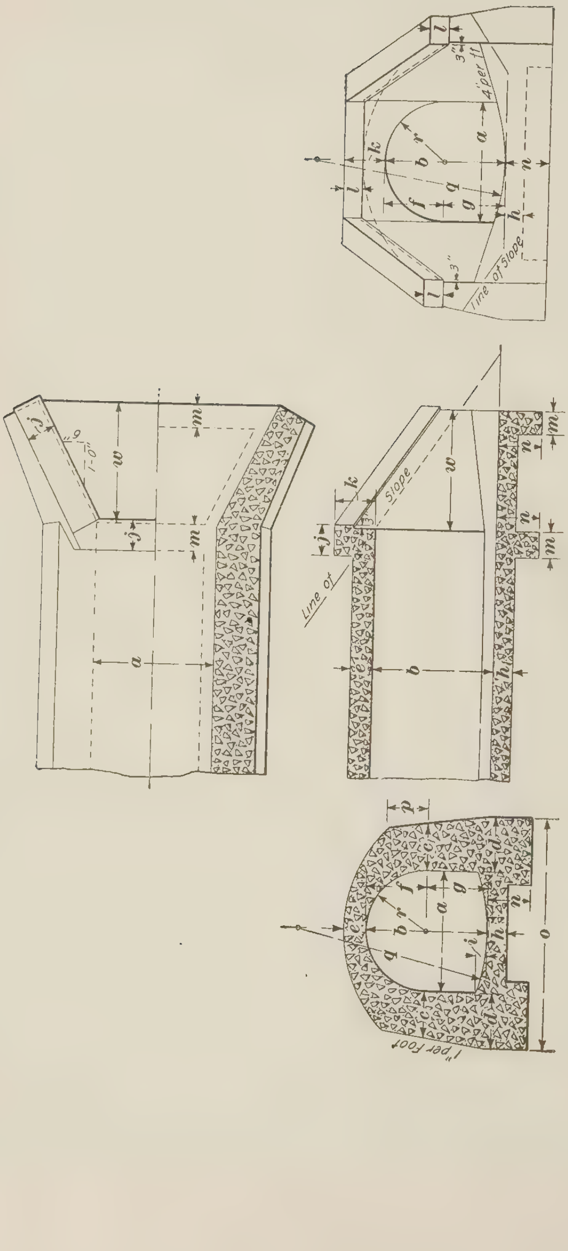
CONCRETE ARCH CULVERTS

92. The concrete arch seems destined to become the common type of culvert, even in localities abounding with good stone. Many engineers batter the inner surface of the side walls of concrete arch culverts below the spring lines, but, unless the span of the arch is increased, this reduces the width of waterway at the bottom, with the result that the total capacity of the culvert is decreased, while the cost is slightly increased.

93. Though in stone box culverts there is little attempt made to protect the structure from the effect of frost, in concrete arches the foundation is commonly carried below the frost line, for the effect of frost may impair the strength of the structure by destroying its monolithic character. The frost line being deeper at the ends of the culvert than where the culvert is partly protected by the bank above, there is greater need of deep foundations under the wings and apron than under the body of the culvert.

94. Fig. 16 shows a typical semicircular concrete arch culvert. The dimensions required for culverts of this form may be found in Table III. Fig. 17 and Table IV give arches with a rise of one-quarter of the span. In the culvert shown in Fig. 16, the invert is curved, while in that shown in Fig. 17 the invert has straight sloping sides. Either style of invert is applicable to both forms of arch.

A comparison of the dimensions for these two forms shows that, for any given span and height of culvert, there is little difference between the semicircular arch and one with a rise of one-quarter of the span (or even one-sixth of the span), either in the waterway or in the required depth of key and thickness of the abutment at the spring line of the arch. As the flattening of the arch raises the spring line, and also increases the rate of spread of the walls, there is a decided increase in the quantity of masonry as the arch is flattened; the maximum of economy and width of structure favors the semicircular arch. For small concrete arch culverts, the most economical design is that in which the height is about equal to the span.


l
3"
4" per ft
k
b
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q
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g
h
n
Line of Slope
3"
l
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w
m
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m
a
k
Slope
w
j
Line of
m
n
n
m
e
b
h
p
c
d
f
g
a
n
e
b
r
i
h
o
q
c
d
1" per foot

FIG. 16

TABLE III

DIMENSIONS FOR CONCRETE SEMICIRCULAR ARCH CULVERTS

(*Portland-Cement Concrete: for Arches, 1 : 2 : 4; for Walls, 1 : 2½ : 6*)

Size of Opening		Thickness of Abutments				Depth of Key e		Rise of Arch f		Height of Wall to Spring Line g		Thickness of Invert h		Dip of Invert i		Width of Parapet and Coping of Wings j		Height of Parapet k		Depth of Coping on Parapet and Wings l		Width of Apron and Cross-Walls m	Depth of Apron and Cross-Walls n	Total Width o		Height of Back of Walls Above Spring Line p		Radius of Invert q		Radius of Crown r		Length of Apron w	Area of Opening x
Width a	Height b	At Spring Line c		At Bottom d																													
Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Square Feet
3	3	2	6	2	7½	1	1	1	6	1	6	1	1		3	2	6	2	7	1	6	2	4	8	3	1	3½	4	7½	1	6	3	7.8
4	4	2	8	2	10	1	2	2		2		1	2		4	2	6	2	8	1	6	2	4	9	8	1	7	6	2	2		4	13.8
5	5	2	10	3	1½	1	3	2	6	2	6	1	3		5	2	6	2	9	1	6	2	4	11	1	1	10½	7	8½	2	6	5	21.6
6	6	3		3	3	1	4	3		3		1	4		6	2	6	2	10	1	6	2	4	12	6	2	2	9	3	3		6	31.1
8	8	3	4	3	8	1	6	4		4		1	5		8	2	6	3		1	6	2	4	15	4	2	9	12	4	4		8	55.4
10	10	3	8	4	1	1	8	5		5		1	8		10	2	6	3	2	1	6	2	4	18	2	3	4	15	5	5		10	86.5
12	12	4		4	6	1	10	6		6		1	10	1		2	6	3	4	1	6	2	4	21		3	11	18	6	6		12	124.6

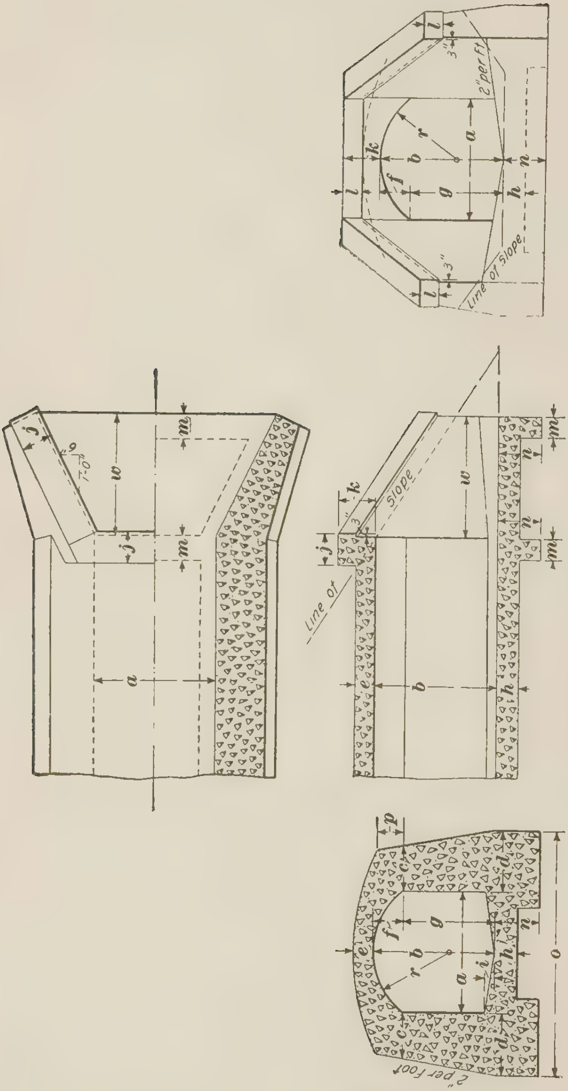
l
3"
2" per ft
r
a
k
b
f
g
h
n
Line of Slope
j
7'-0"
w
m
Line of
Slope
e
c
d
i
o
p
2" per foot

FIG. 17

TABLE IV

DIMENSIONS FOR CONCRETE ARCH CULVERTS WITH RISE = $\frac{1}{4}$ SPAN

(*Portland-Cement Concrete: for Arches, 1 : 2 : 4; for Walls, 1 : 2½ : 6*)

Size of Opening		Thickness of Abutments				Depth of Key e		Rise of Arch f		Height of Wall to Spring Line g		Thickness of Invert h		Dip of Invert i		Width of Parapet and Coping of Wings j		Height of Parapet k		Depth of Coping on Parapet and Wings l		Width of Apron and Cross-Walls m	Depth of Apron and Cross-Walls n	Total Width o		Height of Back of Walls Above Spring Line p		Radius of Crown r		Length of Apron w	Area of Opening x
Width a	Height b	At Spring Line c		At Bottom d																											
Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Square Feet
3	3	2	6	2	10½	1	1		9	2	3	1	1		3	2	6	2	7	1	6	2	4	8	9		11	1	10½	3	8.1
4	4	2	8	3	2	1	2	1		3		1	2		4	2	6	2	8	1	6	2	4	10	4	1	1	2	6	4	14.4
5	5	2	10	3	5½	1	3	1	3	3	9	1	3		5	2	6	2	9	1	6	2	4	11	11	1	3	3	1½	5	22.4
6	6	3		3	9	1	4	1	6	4	6	1	4		6	2	6	2	10	1	6	2	4	13	6	1	5	3	9	6	32.3
8	8	3	4	4	4	1	6	2		6		1	6		8	2	6	3		1	6	2	4	16	8	1	9	5		8	57.4
10	10	3	8	4	11	1	8	2	6	7	6	1	8		10	2	6	3	2	1	6	2	4	19	10	2	1	6	3	10	89.7
12	12	4		5	6	1	10	3		9		1	10	1		2	6	3	4	1	6	2	4	23		2	5	7	6	12	129.2

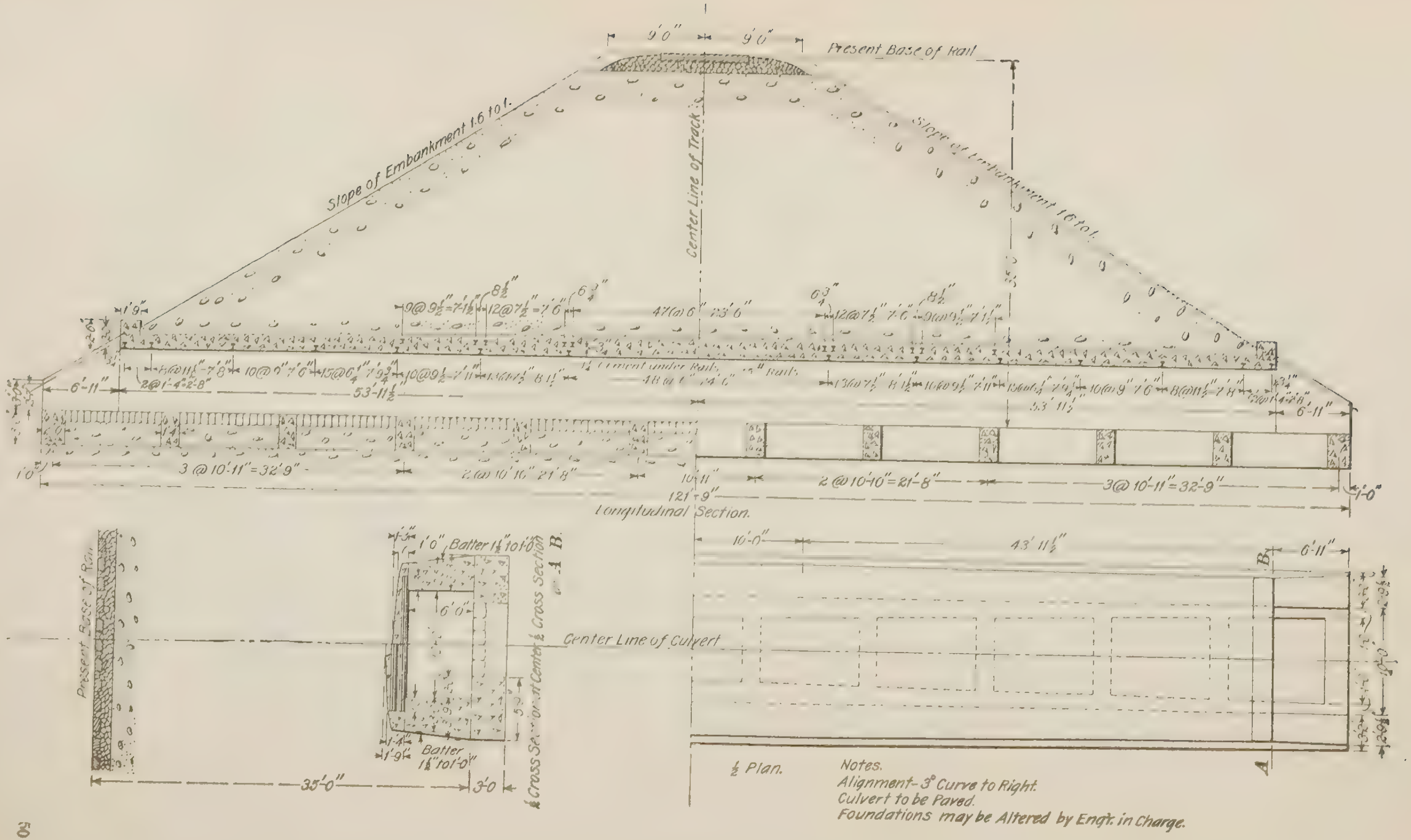
Present Base of Rail
9'0"
9'0"
Slope of Embankment 1.6 to 1.
Slope of Embankment 1.6 to 1.
Center Line of Track
47@6" 23'6"
12@7½"=7'6"
9@9½"=7'1½"
8½"
6¾"
1'9"
6'-11"
53'-11½"
53' 11½"
48@6" 24'6"
3 @ 10'-11"=32'9"
2 @ 10'10" 21'8"
10'11"
121'-9"
2 @ 10'-10"=21'-8"
3@ 10'-11"=32'-9"
1'-0"
Longitudinal Section.
10'-0"
43' 11½"
6'-11"
A
B
½ Plan.
Center Line of Culvert
1'0" Batter 1½" to 1'-0"
6'0"
5'0"
Batter 1½" to 1'-0"
1'-4"
1'-9"
35'-0"
3'-0
½ Cross Section at Center ½ Cross Section A B.
Present Base of Rail
Notes.
Alignment-3° Curve to Right.
Culvert to be Paved.
Foundations may be Altered by Engr. in Charge.

FIG. 18

FLAT-TOP REINFORCED-CONCRETE CULVERTS

95. Concrete box culverts, in which the concrete in the cover is reinforced with steel, seem destined to be very extensively used in the future. A great number of culverts of this class have been built by the Chicago, Milwaukee & St. Paul Railway, in which the reinforcement is made of old rails or I beams. One of these culverts, 10 feet by 6 feet, is shown in Fig. 18, taken from a paper read in 1901 by Albert Reichmann before the Western Society of Civil Engineers.

It will be noticed, in this culvert, that underneath the track and on each side of the center line, for a distance about equal to the height of the bank above the cover of the culvert, two tiers of rails are used; and that for a width of about 24 feet at the center they are placed as close together as they can well be laid and still leave room for any concrete between them. Where the height of bank is less, only one tier of rails is used, and the spacing is increased with the decrease of the depth of fill. A reinforcement of the cover with **I** beams is used in the same way as is shown for rails, and will allow the use of larger spans.

96. Rods also, either plain or in some of the patented forms, are used, as indicated in Fig. 19. The ends of some of the rods are turned up to carry the shear, or short inclined rods may be attached to the main horizontal rod. The longitudinal rods are introduced in the invert to strengthen the culvert for resisting the longitudinal bending moments due to uneven settlement; they may be omitted if the foundation is firm. Where the length desired to be reinforced in either direction exceeds the length of the rods, it is customary to simply lap them over each other a few inches without attempting any other splice than the concrete gives; or the ends of the rods may be looped back to get a better hold on the concrete.

97. Expanded metal or wire netting is similarly used (see Fig. 20), and has the advantage of being already put together; while with rods each piece must be handled separately and held in place during construction. By the use of rods, however, the steel may be distributed more advantageously. Rods or netting are sometimes used in the walls, as shown in Fig. 21, with buttresses *a*, *a* to stay the wider base to the walls.

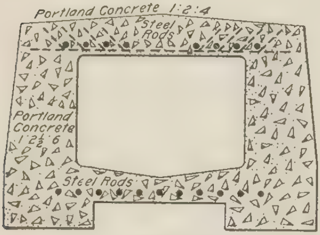

Fig. 19

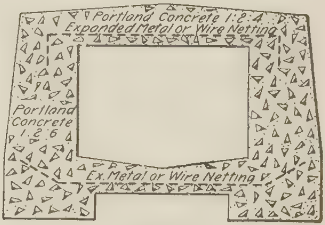

Fig. 20

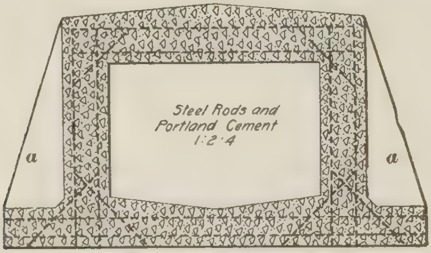

Fig. 21

REMARKS ON CONCRETE WORK

98. For culverts, it is customary to make the excavations for the foundations of walls and pavement (including apron walls and other cross-walls) of just the shape and dimensions desired for these parts; the concrete is tamped into these pits, completely fills them, and takes their form. The top of the invert is shaped by a board drawn over it, a thin layer of rich mortar being added and the top surface of the concrete being finished off by sprinkling on dry cement, which will absorb enough of the moisture from below to acquire the proper consistency. Under where the walls are to be built, the surface of the concrete is left rough; and, unless the forms can be placed and more concrete added before the lower layer sets, it is advisable to place large irregular stones partly embedded in and partly projecting above the top of the concrete, to serve as dowels to bind the new to the old concrete when the work is resumed. While the foundations are being laid and the invert is being finished off, forms and the bracing for them should be prepared in sections of about 10 or 12 feet, so that they can be quickly put in place. If possible, the work of placing the concrete should be resumed before concrete previously laid has had time to take its initial set.

If practicable (and in culvert work it is rarely otherwise), the forms should be so designed that they can be held rigidly in place without rods or any ties passing through the concrete.

99. Fig. 22 shows the cross-section of a concrete arch culvert with the forms in place. The plank lining (called **lagging** or **sheeting**) is usually made of $1\frac{1}{4}$- or $1\frac{3}{4}$-inch plank. The frames of the lower portions and the bracing are of joists from 4 in. × 4 in. to 6 in. × 6 in., according to the size of the culvert. The ribs for the arches are usually made of 2-inch plank well spiked together, with horizontal ties and braces of the same; the ribs are spaced according to the load to be carried, and additional braces may be

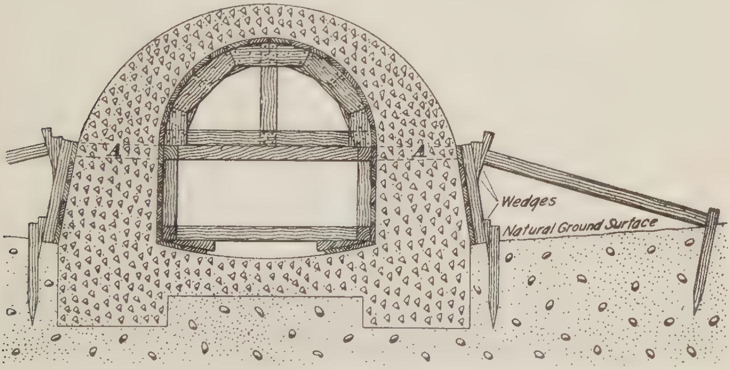

FIG. 22

added if the span requires it. The whole inside form is laid on lagging placed on the invert (which, because of the mode of finishing its surface, hardens more rapidly than the sides under the walls) with wedge-shaped fillers between. The **centers** (as the forms for the arch are called) may be temporarily braced, as indicated by the broken lines at *A A*, until the concrete has reached nearly to that height, when the braces may be removed and the concrete will prevent the moving of the centers. The outer forms may be braced against rocks, if there are any conveniently located, or large stakes may be driven into the ground, and braces footed against them. The lagging should be tongued and grooved, or grooved at both edges and laid with splines, and planed on that side at least against which the concrete is to be laid; it should be matched so true that the outer surface will be smooth. The bracing should be spiked to the forms sufficiently to prevent falling apart, but left so that they can readily be knocked loose when the concrete has set and the forms are to be removed. The top of the structure is finished off with a trowel. In a flat-top culvert, the top should be sloped slightly to drain off any water seeping down through the bank.

Brushing the face of the lagging with soft soap will improve the appearance of the concrete. The lagging is sometimes covered with tin for the same purpose; but as this is expensive, and culverts seldom require much in the way of appearances, such covering may be advantageously dispensed with.

100. There has been a great difference of opinion among engineers as to the amount of water to be used in mixing concrete; but it is now generally conceded that a wet concrete will leave a harder and neater surface, and be more nearly water-tight than a dry mixture, especially if the lagging has been soaped, as suggested in Art. **99.**

When concrete work has to be suspended for so long a time that the old concrete sets before new is added, it is advisable to provide stone dowels, as stated in Art. **98;**

but, whether they are used or not, the old surface should be roughened up and thoroughly wetted before laying new concrete, so as to give the proper bond, and a layer of mortar of neat cement should be plastered on before concreting is resumed.

STANDARD PLANS

101. It is not customary to make detail plans of each culvert, but a **standard plan** is made at the central office for each size of culvert opening; or a general plan is prepared, with the various dimensions represented by letters or other characters, accompanied by a table giving, for each size of culvert contemplated, the figures for each part represented (see Figs. 13, 14, 16, and 17, with their accompanying Tables II, III, and IV). These standard plans usually show simply the cross-section, longitudinal section (and perhaps plan) of the end, and an end view; and may also have directions about modifications to be made for varying conditions. Even though the rules are very explicit, there may be nearly as great a call for good judgment on the part of the resident engineer in these modifications as in determining the location and size of the opening required.

LAYING OUT

102. The required waterway having been determined, the ground should be examined to see whether a good foundation can be obtained for the culvert at the lowest point of the embankment, or whether a decidedly better foundation can be found at a point slightly higher, but within the limit to which it is allowable to impound the water (Art. **3**). The natural watercourse may not be exactly at right angles with the line of the bank, but, if practicable, it is advisable that the culvert should be so placed. If, however, the cost of grading the inlet and outlet will exceed the saving in masonry, more will be lost than gained by squaring up the line of the culvert.

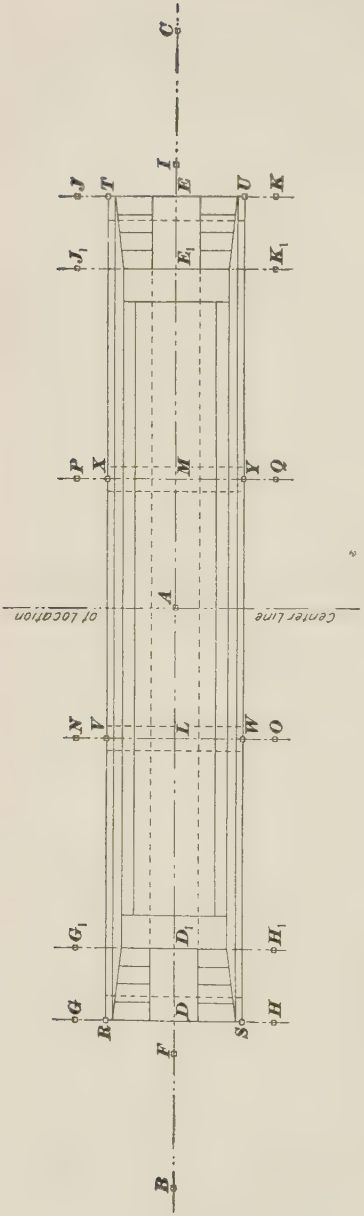

FIG. 23

103. The position and direction of the culvert having been settled, the center line should be marked on the ground by a stake set at the center line *A*, Fig. 23, and one on each side at such points *B* and *C* that they will be safe from disturbance, and so located that all parts of the site may be seen through a transit set over either stake. Next, noting from the plan the relation of the ends of the structure to the slope line, stakes are set at *D* and *E*, where the ends of the paving should be, as determined by the formulas in Art. **80.** From *D*, stakes *F*, *G*, and *H*, in range with the center line of the culvert and at 90° to it, are set at such distances from *D* that they will not be disturbed by the excavation for foundations, and yet will be close enough to be convenient for the use of the workmen. Similarly, from *E*, stakes are set at *I*, *J*, and *K*.

Having ascertained the length of the culvert, the number of cross-sills is readily determined and their location fixed, as at *L* and *M*, and stakes *N*, *O*, *P*, and *Q* are set to mark the range of their axes. It is advisable, if practicable, to place the stakes *F*, *G*, *H*, *I*, *J*, *K*, *N*, *O*, *P*, and *Q* all at the same distance (say 3 feet) from the edges of the foundation. All these stakes should be firmly driven, and the exact range marked on them by tacks; each stake should be marked to indicate plainly for what it is intended to give line; as, ℄, or *C. L.* (center line) for stakes *F* and *I*; *End* for *G*, *H*, *J*, and *K*; and *X wall* for *N*, *O*, *P*, and *Q*. The height of the top of the pavement opposite, above, or below the level of the top of the stake should also be marked on the side. If desired, special grade stakes may be set to give the level of the pavement; if such special stakes are set, they should be cut to a different shape from those used for ranges. Stakes should also be set at *R*, *S*, *T*, *U*, *V*, *W*, *X*, and *Y*, at the edges of the excavation, and left high so that they may be readily seen; these stakes have no tacks, and may be displaced as soon as the excavation is begun.

Some foremen prefer to have the ends of the cover staked out, and make their own allowance for the wings, instead of having stakes for the ends of wings and making allowance for the cover. In this case, stakes are set at G_1, H_1, J_1, and K_1, instead of at *G*, *H*, *J*, and *K*.

ESTIMATE

104. As culverts are usually built, the cost, and sometimes also the contract price, for the pavement of stone culverts is less than for the walls and cover. The parapets and wings, or perhaps simply the coping stone, are also frequently classed separately from the rest of the culvert. If the culvert is founded on rock for the whole or a part of its length, there need be no other pavement, but all loose and scaly rock must be removed, and the surface of the rock must be leveled off to give a suitable bearing for the walls. The surface of the rock in such cases is usually so irregular that the height of the wall will vary considerably; and, the capacity of the culvert being limited by its minimum opening.

these varying heights increase the amount of masonry in the walls. Standard plans are generally accompanied by a schedule of the quantity of masonry of each class per running foot of culvert, with a stated addition for the parapet, wings, and apron at each end. It is customary with some engineers to make estimates for the contractors from these figures, disregarding the changes that may be caused by the procedure; in the average of cases, this will work no hardship on either party. A better way, however, is to make proper allowance for the preparation of the rock and for the extra wall required, and omit any estimate for paving where no paving is actually placed. In this way, the price paid for each structure will properly represent its reasonable cost, without relying on an over-payment for one structure to offset the under-payment for another. If there are different contractors, *A* will hardly be soothed for his losses by knowing that *B* has made a large profit.

TUNNELS

TUNNEL SURVEYING

TUNNEL ECONOMICS

1. Importance of Careful and Accurate Work.—The location and the construction of tunnels are so intimately connected that it is considered best to treat them together instead of making tunnel location a part of railroad location. The cost of constructing tunnels is comparatively so great, and the consequences of a blunder so far reaching, that an error in location is a very serious matter. It is, therefore, of the utmost importance that every phase of the location should be studied with especial care, and that special pains should be taken to perform the surveying work so as to obtain exceptionally accurate results.

2. Choice Between a Tunnel and an Open Cut.—An engineer frequently has to decide for himself whether, under certain conditions, a tunnel is economically preferable to a cut. This question is so broad that only a general idea of the method of solution can be given here. There are three general cases in which a tunnel is preferable; namely, (1) when the depth is so great that an open cut is out of the question; (2) when the tunnel is merely a subsurface tunnel, the surface being utilized for other purposes; (3) when the depth is such that it is cheaper to tunnel than to make an open cut.

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In the first case, there is no alternative except, possibly, an utter change in the line of the road for a considerable distance. The second case is usually decided by considerations that have little or no relation to engineering. The third case is purely an engineering question. It is generally assumed that, when the depth of cutting exceeds 60 feet, it becomes expedient to drive a tunnel; but, unfortunately, the problem cannot always be solved by so simple a rule. A very soft soil, when dug out for an open cut, requires comparatively flat side slopes and therefore an enormous volume of cut when the center depth is great. A tunnel in such soil will be easy to dig, but will require expensive and strong timbering. An open rock cut may be made with steeper side slopes and much less volume of cutting. A tunnel through such rock may require little or no timbering, but the rock will be difficult to blast out. The requirements for material for fill in the vicinity of the location have considerable influence on the question. If a large amount of material is required to make a near-by fill, it will be justifiable to make an open cut to a depth much greater than that at which a tunnel would otherwise be more economical.

3. **Approaches.**—The open cuts at the ends of a tunnel are called **tunnel approaches.** The engineer must decide when it is expedient to discontinue open cutting and begin tunneling. The principles regarding relative cost of an open cut and of a tunnel must again be used. Where the slope of the hill that is tunneled is very steep and the line approaches the hill in the direction of the steepest slope, the length of the approaches will probably be very short. On the other hand, when the hill slopes are comparatively flat, there may be a very long open cut before reaching a depth where a tunnel would be either economical or practicable. Even the tendency of the approaches to fill with snow in winter, or to be blocked by landslides, may carry some weight in deciding on the better plan to adopt.

4. **Alinement and Grade.**—There are many economical reasons, from the standpoint of both construction and

operation, why a tunnel should be built straight. This, however, is sometimes impracticable, except by the adoption of a design that would be too expensive in other respects. When a tunnel forms a link in a long maximum grade of the road, there is the temptation to continue that grade through the tunnel; but it should be remembered that the slipping of the driving wheels on the wet rails in a tunnel, as well as the increased atmospheric resistance, reduces very materially the capacity of a locomotive. The grade in a tunnel should, therefore, be materially reduced below the grades outside. It is often preferable to reduce the grade so as to make it as nearly level as is consistent with proper drainage, say .2 per cent. If the tunnel is very long, it is preferable to have two grades, each rising from the ends toward the center. Where the tunnel must form part of a long grade, the grade may be all in one direction. Ventilation will be somewhat assisted by having the grade all in one direction. In constructing a long tunnel on the Great Northern Railroad, an allowance of .5 per cent. in the grade was made. The tunnel formed part of a grade of 2.2 per cent., and the part in the tunnel was reduced to 1.7 per cent.

LINES AND LEVELS

5. Preliminary Operations.—The decision to construct a tunnel at a given locality implies a previous surface survey of the entire length of the proposed tunnel. When the location of the tunnel has been definitely determined, the next step is a very precise resurvey of the line of the tunnel on the surface. Tunnels are usually constructed by excavating from each end, and sometimes by excavating in each direction from the foot of shafts; this requires the most minute accuracy in all the surveying operations, so that when the headings meet there will be no appreciable discrepancy in the two or more surveyed lines, either laterally or vertically. If the tunnel is very short, the ordinary engineers' transit, when properly handled, will give as close results as are necessary. When the tunnel is very long, it is almost essential to have a special instrument, called a **tunnel transit,** by which a great degree of accuracy can be obtained. The telescope of a tunnel transit is unusually large and powerful; it can be taken out and inverted in the wyes, which are open, and is fitted with a striding level by which the transverse axis can be made truly horizontal. Whatever instruments are used, they should be tested often and kept in perfect adjustment.

The approximate line of the tunnel is first run with ordinary instruments, by means of which a series of points is determined, each intermediate point being so located that the two points between which it lies are plainly visible from it. In Fig. 1, *e p q n g* represents a section of a hill to be tunneled, and *a*, *b*, *c*, *d* are points determining the line of survey. Intermediate points *h*, *p*, *k*, *q*, etc. are also marked. The

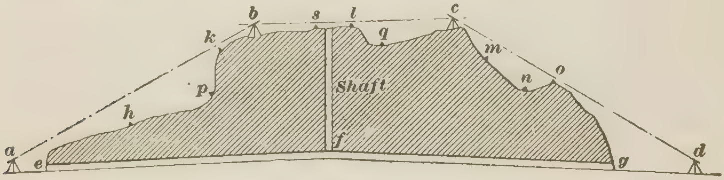

FIG. 1

points *a*, *h*, *p*, *k*, *b*, etc. should be marked by means of large plugs firmly set in the ground; these plugs may be made of 6″ × 6″ timber. By means of repeated observations, in which all errors of adjustment are carefully eliminated, points that are exactly on a line may be determined on the tops of these plugs. The plugs (or "monuments") should be set into the ground so far that they will not be affected by frost. It is wise to fill the pit around these monuments with broken stone or even concrete, well rammed in. The work of the surface survey is preferably done in calm, cloudy weather. The early morning is the best time for this kind of work, provided that the wind is not blowing; the errors due to refraction are then least. Some engineers prefer to do the work at night, using plummet lamps to sight on. The center surface lines of several long tunnels now in operation were run in this manner.

As shown in Fig. 1, points have been established not only at *a*, *b*, *c*, and *d*, but also at several intermediate points. These intermediate points may be utilized when the points at a greater distance are temporarily invisible on account of fog. By setting up a transit at *a* and *d* and obtaining alinement from some one of the previously established points, intermediate alinement points may be rapidly located.

6. Levels.—A line of levels must be run over the surface, determining the elevations of all points. There is nothing peculiar about this leveling, except that, since the slopes are usually very steep, it will require unusual care to avoid error. As far as possible, the foresights and backsights must be made nearly equal, so as to neutralize errors in the adjustments of the level.

7. Determination of Horizontal Distances.—Horizontal distances may be determined in one of two ways.

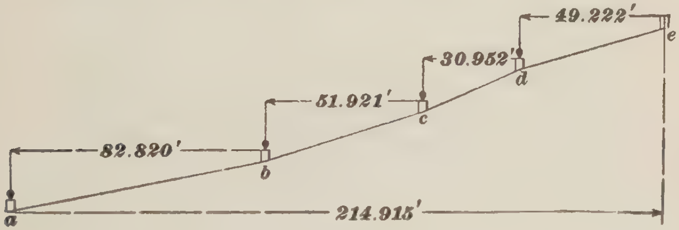

Fig. 2

The usual method of making precise tape measurements, by holding the two ends of the tape constantly level, as illustrated in Fig. 2, may be followed. This method will serve for measuring the line over the comparatively level section that is sometimes found over the middle part of the tunnel (as from *b* to *c*, Fig. 1); but near the ends of the tunnel the slopes are usually so steep that it is impracticable to hold any great length of tape truly horizontal. The use of this method in such cases will require that the plugs shall be placed comparatively close together, and that the position of the end of the tape over each plug shall be plumbed down with a plumb-bob. While this method is theoretically precise, there is a certainty of a small inaccuracy at each point, and the accumulation of a very large number of these, although some of them will counterbalance, will make the final result somewhat uncertain.

8. For very precise work, the length of the tape must be corrected for temperature. The length of a steel tape increases by about .0000065 of itself for every degree Fahrenheit that the temperature rises, and decreases by an equal proportional amount for every degree Fahrenheit that the temperature drops. Thus, if the tape has a length of 100 feet at 60°, its length at 90° will be

$$100\,[1 + (90 - 60) \times .0000065] = 100.02 \text{ feet},$$

and its length at 25° will be

$$100\,[1 - (60 - 25) \times .0000065] = 99.98 \text{ feet}$$

Another thing that must be taken into account is the force with which the tape is pulled. The cross-sectional area of a fine steel tape may be as small as .002 square inch. Assuming that the modulus of elasticity of the tape is 30,000,000 pounds per square inch, this means that a pull of 15 pounds will stretch the tape $\frac{1}{4000}$ of its length, or .025 foot, which is about .3 inch. Besides, when the tape is swung clear from the ground, as is usually necessary in field operations, the sag of the tape is equivalent to an increase in its length, since the distance between its two ends, as registered, is evidently greater than the true distance. Elaborate calculations are sometimes made to determine the true distance between the ends of the tape under a given tension. Usually, such calculations can be, and are, avoided by determining what is called the **normal tension** on the tape. The normal tension is such a tension that the increase in length of the tape due to its stretch is exactly compensated by the loss in horizontal distance due to the sag. This normal tension can be obtained without any calculations by stretching the tape on a level floor with just enough tension to make it taut, marking its length on the floor, and then raising the tape clear from the floor and noting by means of a spring balance and plumb-bobs just what tension is required to make the horizontal distance between the ends the same as before. In making this test, the greatest care must be taken that the ends of the tape are perfectly steady, and that the plumb-bobs are used to project vertically the exact position of the end marks downwards to the floor.*

9. The second method of determining horizontal distances, and the only one that can be conveniently used on very steep slopes, is illustrated in Fig. 3. The inclined distances *a b*, *b c*, etc. are measured directly with the tape, and then the differences of elevation, or vertical distances *a a′*, *b b′*, etc., are determined from the level notes. The true horizontal distance may be determined for each section: (1) by subtracting the square of the vertical distance from the square of the inclined distance, and extracting the square root of the difference; (2) by the methods employed in trigonometry for the solution of right triangles.

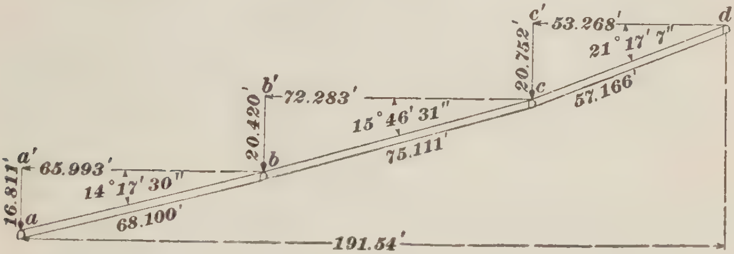

FIG. 3

EXAMPLE.—The slope distance *a b* was found to measure 68.10 feet; the difference of elevation between *a* and *b* was found from the level notes to be 16.811. What was the horizontal distance *a′ b*?

SOLUTION.—The right triangle *a b a′* gives

$$a'b = \sqrt{ab^2 - aa'^2} = \sqrt{68.10^2 - 16.811^2} = 65.993. \quad \text{Ans.}$$

Otherwise, thus:

$$\sin aba' = \frac{16.811}{68.10}; \quad aba' = 14° \, 17' \, 30''$$

$$a'b = 16.811 \div \tan 14° \, 17' \, 30'' = 65.993. \quad \text{Ans.}$$

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*Further details relating to temperature and tension corrections are given in *City Surveying*.

EXAMPLES FOR PRACTICE

1. Assuming the inclined and vertical distances given in Fig. 3, verify the values for $b'c$ and $c'd$.

2. The inclined distance between two plugs is 29.746 feet. The elevations of the two plugs above datum are, respectively, 196.349 and 213.073 feet. Find the horizontal distance. Ans. 24.599 ft.

10. Curved Tunnels.—Curved tunnels should be avoided where practicable; but, as may readily be seen by the case illustrated in Figs. 4 and 5, it is sometimes impossible to avoid them. In such a case, the tangent distances are calculated and the P. C. and P. T. are located on the surface by direct measurement. The work and calculations are repeated many times, and every possible precaution is taken to secure perfect accuracy of results. The sketch given in Fig. 4 shows the difficulties attending the laying out of the Rockport tunnel on the Lehigh Valley

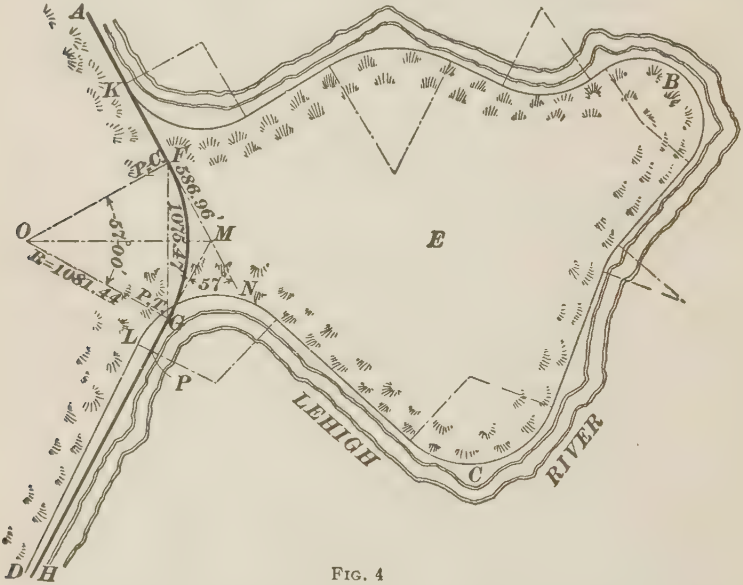

FIG. 4

Railroad. The original line *A B C D* followed the course of the Lehigh River, which hugs the bluff *E*. The tunnel line *A F G H* would have been adopted, and the tunnel driven when the road was first constructed, but a rival line was building on the opposite side of the river, and there was a race to reach the Wyoming Valley coal fields and command the coal traffic. The tunnel line was accordingly postponed, and the river line adopted. The tunnel was driven in 1882-83, after a lapse of 20 years. The neck *F G* through which the tunnel passes (a profile of which is shown in Fig. 5) reached a height of more than 300 feet. The hillsides were so steep that in places a man could hardly stand. The grade of the original line *A B C D* is about 20 feet per mile, rising from *A* to *D*, and the length is 1½ miles greater than that of the tunnel line *A K F G P*. The grade on the new line from *K* to *P* was made 20 feet per mile, the same as the grade of the tangent *A K* of the old line; this brought the point *P* 30 feet lower than the point *L*. The new line *P H* was then given a grade of 40 feet per mile so as to gain on the old line *L D* at the rate of 20 feet per mile, coming to the same elevation 1½ miles from *L* and *P*. The new tangent *P H* was then located so as to meet *L D* at a distance of 1½ miles from *L* and *P*. The intersection angle *G M N* measured 57°, and a 5° 18′ curve was decided on; the tangent distances *F M* and *M G* were then measured directly on the surface, and the P. C. and P. T. were set.

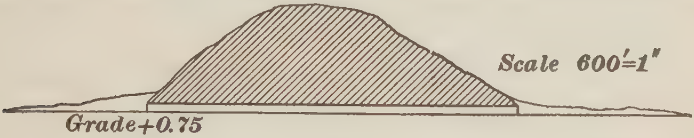

FIG. 5

On account of the steepness of the slopes and the height of the hill, much difficulty was experienced in making a satisfactory intersection to locate the point *M*. Within a distance of 500 feet, there was a difference in elevation of more than 300 feet, and some of the sights contained a vertical angle of more than 60°. The lines were run principally in the early morning hours, though some of the best results were obtained on cloudy days. A large tunnel transit with powerful lenses, and of more than double the weight of an ordinary transit, was used. Common pins against a dark background were used for backsights. First, an intersection was made, large plugs or hubs 6 inches square being used. The tangent *K M*, Fig. 4, was then repeatedly run, and each line marked on the hubs *Q* and *S*, Fig. 6, the hubs being so located that the intersection *M* was between them. Each line was marked with tacks, each of which was numbered, as shown in the figure. The lines varied each time, no two coinciding. One or two fell wide of the mark, and were ignored. Finally, the mean of the lines was adopted. The tangent *G M*, Fig. 4, was then run an equal number of times, and each intersection on the line *Q S*, Fig. 7, marked on the hub *M* with a tack and numbered. The mean of these intersections was taken as final.

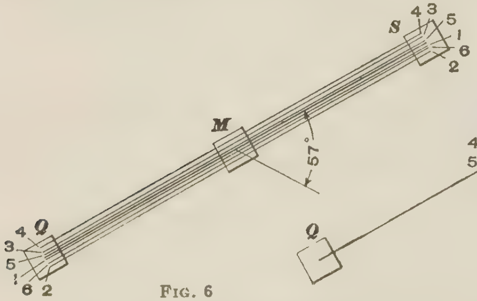

Fig. 6

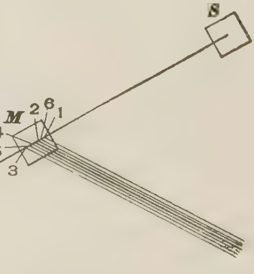

Fig. 7

Equally great difficulty was experienced in locating the P. C. and P. T. The distance was measured many times, and each distance marked. The mean was then taken as the correct measurement. The top of the hill had the form of a plateau, and the center *O*, Fig. 4, of the curve was located by turning a right angle to the tangent *K F* at *F*, the P. C., and measuring the radius, 1,081.44 feet. The central angle *F O G* of 57° was then turned, and the second radius *O G* run out and measured. The line and measurement falling on the plug at the P. T. at *G* proved the work correct. The result of all this care and pains was that, when the tunnel was driven from both ends and the headings met, there was less than $\frac{1}{2}$ inch discrepancy in the two lines.

11. Interior Surveys.—During construction, the alinement and levels for the interior of the tunnel must be located, relocated, and tested with great frequency. Surveying points in the interior of the tunnel must be placed where they are not likely to be disturbed or covered. In general, the points that are least likely to be disturbed are in the roof of the tunnel. Points are usually located by drilling a hole into the roof and filling it with a plug of wood driven in very tight. Into this plug is driven a screw eye, or else a **spud,** which is a nail having a round flattened head with a hole through its center (see Fig. 8). Sometimes, the screw eyes or spuds are driven in the timbering of the tunnel; but, as the timbering is likely to be jarred out of place, such locations should be tested very carefully and as frequently as possible, especially when the line is being carried forwards. A transit can be centered under a roof point by setting the instrument in such a position that the center of the telescope will be directly under the point of a plummet suspended from the roof point. A transit telescope usually has a small hole drilled in the top of the ball that forms the connection between the telescope and the axis. When the telescope is horizontal, this hole is exactly over the center of the instrument, and the transit can easily be placed so that the hole is directly under the plumb-bob point. Another method is to determine, by means of a plummet, a temporary point on the floor of the tunnel directly under the spud, and then center the transit over that point in the usual manner. This method requires two operations instead of one, and usually is not so good as the other.

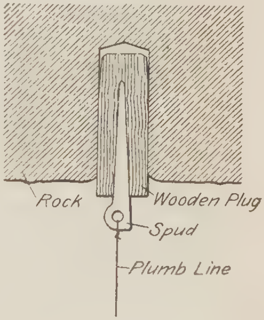

FIG. 8

12. In underground work, it is always necessary to illuminate the cross-wires of the transit or level. This may be done by holding a bull's-eye lantern so that its light will shine into the telescope and yet be outside of the direct line of vision. A preferable method is to use a reflector, such as is illustrated in Fig. 9. In this case, the lantern may readily be held by the transitman (or preferably by an assistant) in such a position that its light will be reflected from the reflector directly back on to the cross-wires, but without interfering with the line of sight. For leveling work, the face of the level rod must be illuminated sufficiently to determine the readings. A transit pole or a plumb-line may similarly be illuminated as a target for the transit work. A preferable method is to use a plummet lamp, such as is illustrated in Fig. 10. This lamp has an oil reservoir of brass, with a steel point at its lower end. A cylindrical stem contains a wick at the upper end. The lamp is suspended by gimbal points to a bail, to which a wire is attached. This wire can be run through the eye of a spud or screw eye, and the plumb will then be vertically under the spud. To locate a point on a given line by means of such a plummet lamp the lamp is suspended by the wire and shifted laterally until the line of sight bisects the flame. If, at the same time, the upper end of the wire is held against the roof of the tunnel, the point in the roof that is in the desired line is readily determined and may be marked with a plug, as previously described.

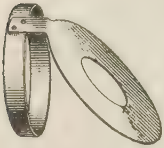

Fig. 9

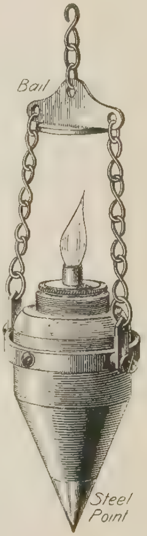

Fig. 10

Grade points, also, may be located in the tunnel by means of these plugs in the roof. It is only necessary to invert the level rod so that its zero end is against the point in the roof. The rod reading then indicates the distance of the cross-wires below the point in the roof. To determine the elevation of the plug, the rod reading must be added to the height of the instrument, instead of subtracted, as in ordinary level work. Roof plugs may be used also as bench marks, but bench marks are sometimes made by drilling a hole horizontally into the side of the tunnel at some convenient height, into which plugs of wood or iron are firmly driven. These plugs project far enough from the wall to allow a level rod to be held on them in a vertical position. In a timber-lined tunnel, large nails or spikes may be driven horizontally into the wall posts and serve the same purpose, although there is some danger that the timbers may shift their position sufficiently to destroy the accuracy of the work. These points should, therefore, be frequently tested.

13. If the tunnel is curved, it is necessary to determine the true center line at very frequent intervals, in order that the cross-section of the tunnel may be sufficiently large at each point, and yet not excessively or wastefully so. The location of the points of the curve is effected by the methods explained in *Circular Curves*.

It should be noted that, since it is easier in tunnel work to locate points on a tangent and then make some offsets, it is better to locate a number of lines that are all tangent to the curve and that have their intersections mutually intervisible, which of course means that they all lie within the cross-section of the tunnel. The lengths of these tangents and the angles between them can be easily computed. The desired points of the circular curves may be located by offsets from these tangents.

14. Surveying Down a Shaft.—In order that the excavation at the bottom of a shaft may be properly directed, it is necessary to transfer from the surface to the bottom of the shaft the three surveying elements of *distance*, *elevation*, and *direction*. Since the shafts are almost invariably made vertical, it is possible to suspend a heavy plumb-bob by means of a wire of sufficient length from the top to the bottom of the shaft. The element of distance, which in this case means the horizontal distance of any given point from a reference point, is very readily transferred, with sufficient accuracy, from the top to the bottom of the shaft by means of the plumb-bob. Similarly, the difference of elevation may be measured with all necessary accuracy by merely measuring vertically down the shaft with a steel tape. The third element, that of direction, requires great care, and often taxes the skill and ingenuity of the locating engineer. In its bare outline, the method is simple and perhaps looks easy. Practically, the chances for inaccuracy are large and precision is very difficult. The details of the method generally used are as follows: Two pieces of plank C and D, Fig. 11, are spiked to the shaft timbers where the center line of the surface survey crosses; the edges of the plank project inwards

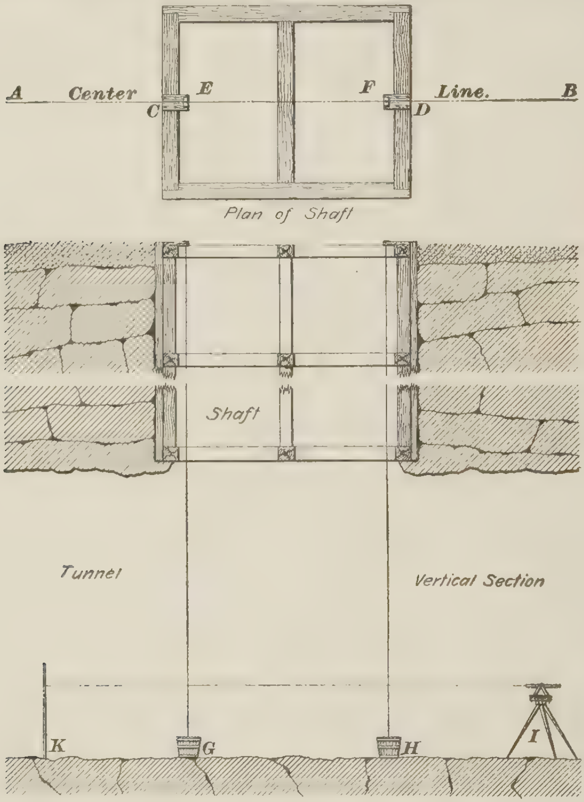

FIG. 11

over the shaft wall. Slots are cut in the edges *E* and *F* of the planks *C* and *D* on the center line. An iron plate with a carefully drilled hole in its center is placed over each slot and in the center line of the surface survey. Holes are drilled in the corners of the plates, which are fastened to the plank. Plumb-bobs weighing from 20 to 30 pounds are suspended by fine steel wires that pass through the holes in the plates. In order to prevent the vibration of these wires, each plumb-bob is entirely immersed in a pail of oil placed at the bottom of the shaft. When the plumb-bobs come to rest, the lines that suspend them are exactly in the center line, as laid down on the surface of the ground. The tunnel having been driven some distance from the foot of the shaft, a transit is set up in it at some point *I* as far as possible from the wires that carry the bobs at *G* and *H*, and moved until its line of sight coincides with the line determined by the two wires—that is, until the vertical wire of the transit covers both of the wires that carry the bobs. A plug is set on line at some point *K*, the instrument is moved to *K*, and a backsight taken on *I*, the latter point having been previously marked by a plug; the line from *K* to *I* should intersect both supporting wires. As the points *E* and *F* are usually not more than about 8 feet apart, and as the heading may run several thousand feet from the shaft before meeting the heading from the other direction, a very slight error in the location of the points *E* and *F* will be greatly magnified. It is also found that drafts of air, which are invariably found in a shaft, prevent the wires from hanging truly vertical, and hence the horizontal line passing through the wires at the bottom of the shaft may or may not be parallel to the surface line. When the heading from a shaft meets the heading from another shaft, or from one end of the tunnel, the alinement should be verified and corrected as much as necessary, even though the correction requires a slight change in alinement.

TUNNEL CONSTRUCTION

CROSS-SECTIONS

15. Single-Track Tunnels.—The usual general form of cross-section for a railroad tunnel consists of an arched roof with vertical or nearly vertical side walls. The details, however, vary according to the character of the material excavated. The dimensions of a tunnel section vary according to the form of section and the number of tracks in the tunnel. For a single-track tunnel, the following dimensions are considered suitable: in hard rock with no lining, from 14 to 16 feet wide at the bottom; 16 feet wide at from 14 to 16 feet above the bottom; and the roof arch on a curve of 8 feet radius. The same general dimensions are suitable for a single-track tunnel lined with timber or masonry in the ordinary manner; in this case, the tunnel excavation is made sufficiently large to allow for the thickness of the lining outside the regular tunnel section. In Fig. 12 is shown a good form of unlined tunnel section, such as is adopted for solid unstratified rock,

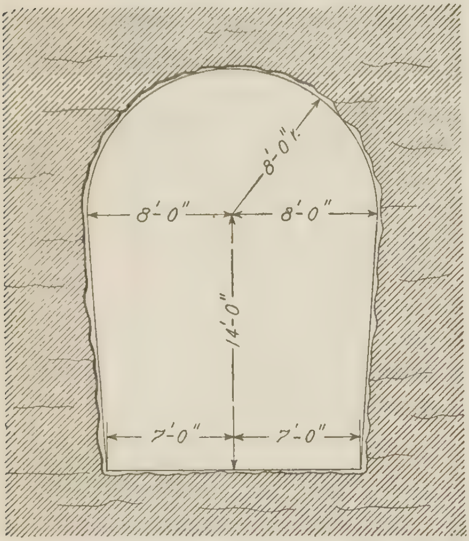

Fig. 12

where there is no danger of slips or falling rock. In **Fig. 13** is shown a good form of section where the tunnel is lined with concrete.

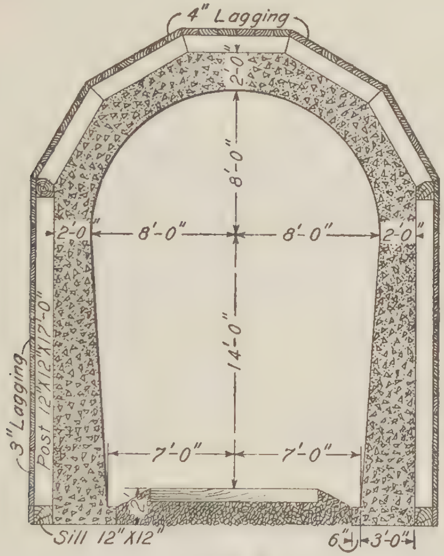

FIG. 13

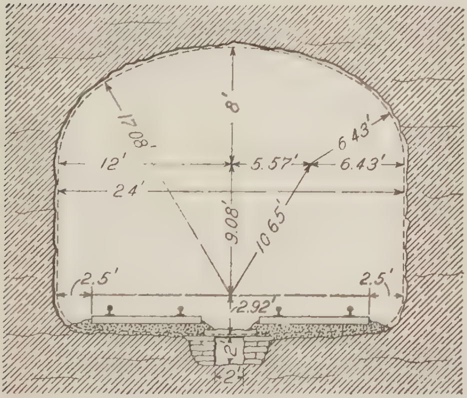

FIG. 14

When a tunnel is built on a curve, it is customary to make the section a little wider than that for a straight tunnel.

16. Double-Track Tunnels.—For a double-track tunnel, the form of section shown in Fig. 14 may be used. In some cases, instead of a three-centered arch, as shown in the figure, the roof arch consists of a semicircular curve of 13½ feet radius.

17. Special Forms of Tunnel Sections.—In some kinds of material, it is necessary to use special forms of sections in order to permit a suitable form of lining to resist the pressure on the roof and sides of the tunnel. In Fig. 15 is shown a form of section used in a portion of the Aspen tunnel on the Northern Pacific Railroad. The interior dotted outline represents the cross-section of a passenger car and shows the amount of clearance between the sides of the car and the walls of the tunnel.

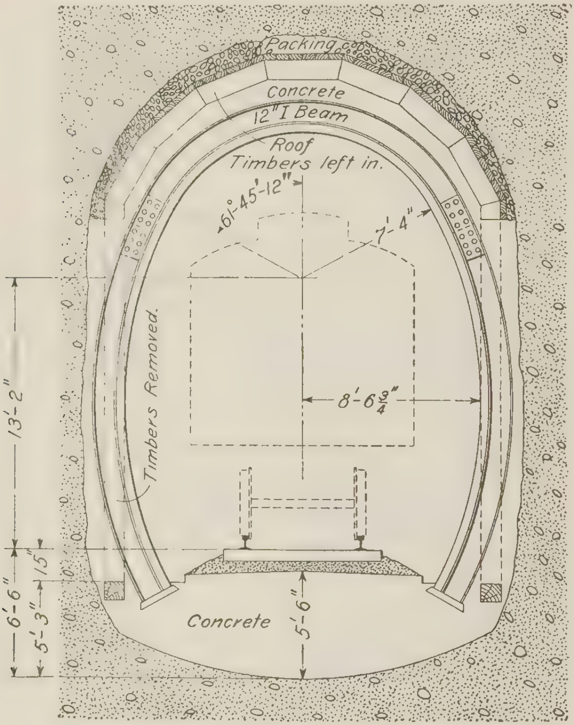

FIG. 15

METHOD OF CONSTRUCTION

18. Headings.—The almost invariable method of driving a tunnel is to excavate first a **heading** (see Fig. 27) whose cross-sectional area is about one-fifth or one-fourth of the total cross-section of the tunnel. The rest of the cross-section is called the **bench.** Frequently, the heading is driven through the upper part of the cross-section, as this renders it somewhat easier to place the timbering during enlargement. If the ground is very wet, there is an advantage in making the heading near the bottom of the section, as then the bottom of the heading will act as a drain, and the soil will be much drier and firmer by the time the enlargement is commenced. The heading is usually made about 5 feet wide and from 6 to 8 feet in height. The face of the heading is usually kept at least 50 feet in advance of the bench (see *B* and *B'*, Figs. 26 and 27). Sometimes, two headings are driven along the lines that will ultimately be the two lower corners of the tunnel section.

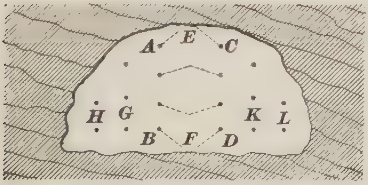

FIG. 16

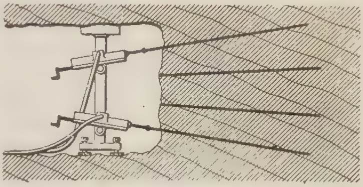

FIG. 18

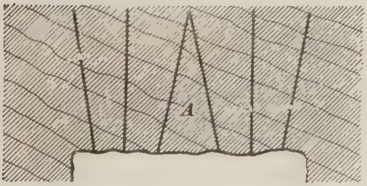

FIG. 17

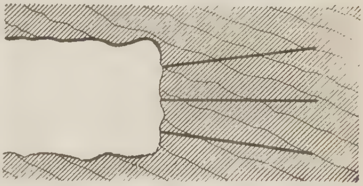

FIG. 19

When the material encountered is a hard self-sustaining rock, the work of excavation is comparatively simple, and, on account of the fact that there is no necessity of timbering, the work is actually less expensive than for very soft material, which must be heavily timbered. This work is usually done with power drills worked with compressed air. The plant for operating these drills will be considered in a subsequent article. Drills working in headings are usually mounted on columns, two drills on each column (see Fig. 18). The drills working on the bench are usually mounted on tripods (see Fig. 26). The air pipe is carried to within about 50 feet of the bench, where a bench hose of equal diameter is attached to it. At the end of the hose is a metal nozzle, called a **manifold,** containing hose connections for each of the drills.

A section of heading showing the arrangement of drill holes in the face is given in Fig. 16. The two middle rows of holes AB and CD converge at an angle of about 20°, meeting nearly on the centre line EF of the tunnel; they are called the **center-cut holes.** The mass of rock included by these holes is wedge-shaped, as shown by the plan in Fig. 17. The removing of this wedge by blasting is called **breaking the cut.** Fig. 18 shows a longitudinal section through the center-cut holes. The rows of holes G, H, K, and L, Fig. 16, on each side of the center-cut holes are called **side rounds.** If there is but one row on each side, the rows are called **single side rounds;** if two rows, **double side**

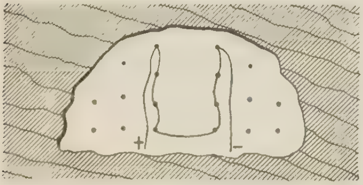

FIG. 20

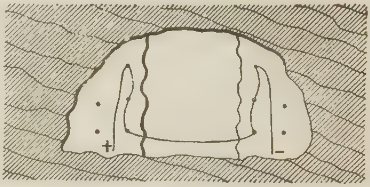

FIG. 21

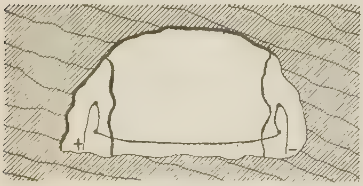

FIG. 22

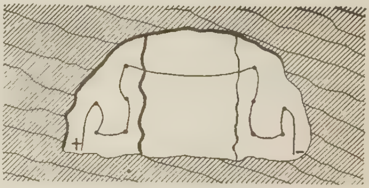

FIG. 23

rounds. A longitudinal section through the side holes is given in Fig. 19. The cut and the side rounds are loaded at the same time. The cut is fired first (see Fig. 20). The side rounds are either fired separately—that is, one row on each side of the cut at a time (see Figs. 21 and 22)—or they are double-fired—that is, both rows simultaneously, as shown in Fig. 23. In Figs. 20 to 23 the irregular lines that connect the holes represent wires through which electric currents are passed to ignite the blast.

19. Enlarging the Heading.—In that portion of the heading shown in the figures just mentioned, the holes are drilled directly into the face of the heading. After they are fired and the material is removed, side holes are drilled at an angle of about 60° with the center line, denoted by the letters *CL*, as shown in section in Fig. 24 and in plan in Fig. 25.

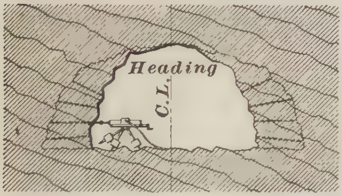

FIG. 24

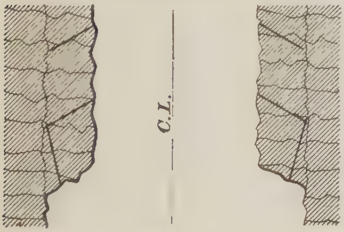

FIG. 25

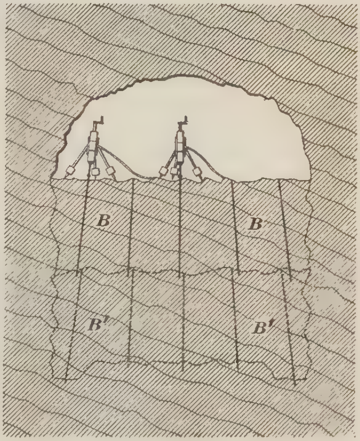

FIG. 26

20. Removing the Bench.—The bench is taken out in two sections, B and B', as shown in section in Fig. 26. The full tunnel section is shown by dotted lines.

The holes in the bench are inclined backwards from a vertical line. A longitudinal section through the center line, showing the usual mode of drilling headings and benches, is given in Fig. 27. The center-cut holes in the heading *H* and all the bench holes at *B* and *B′* are usually fired together, and then the double side rounds in the heading. The center cut offers the greatest resistance to blasting. The holes are consequently loaded with more powerful explosives than are used for either side rounds or bench.

In driving the New York aqueduct tunnel, the cut was loaded with dynamite containing from 60 to 80 per cent. of nitroglycerine, while the average bench powder contained but 40 per cent. of nitroglycerine. On some sections, where rock of special hardness was encountered, the cut was loaded with pure nitroglycerine. This operation, however, is always attended with great danger. After several premature explosions, resulting in considerable loss of life, the use of pure nitroglycerine was abandoned. The effect of firing the cut is generally to pulverize the rock, and all tunnel blasting is intended so to break the rock as to render the use of sledges unnecessary in reducing masses of rock to sizes convenient for loading.

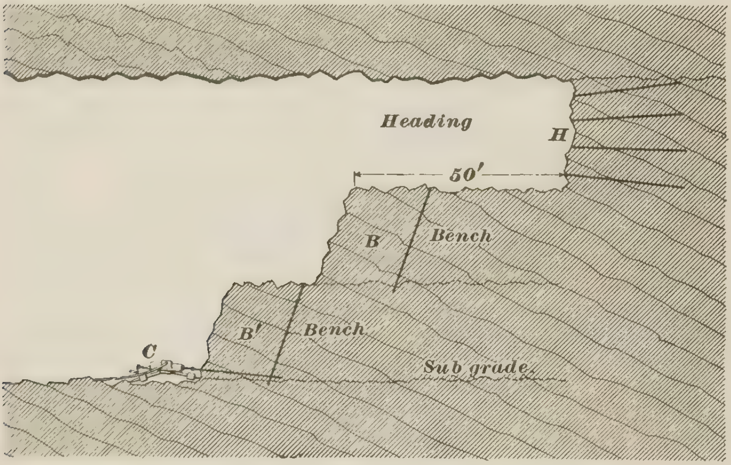

FIG. 27

The efficiency of the powder depends largely on the judgment used in locating the holes and on the angle at which they are bored. The position of the machine while drilling holes at the foot of the bench is shown at *C*, Fig. 27.

21. Power Plant.—The plant for furnishing the power used in driving a tunnel will vary in size according to the length of the tunnel. For a tunnel of moderate length, a single plant located near one of the portals will usually provide all the power necessary to run the machinery for drilling and pumping that is used in both headings, and also to ventilate the tunnel. For a tunnel of considerable length, it is usually better to install a power plant at or near each end. In short tunnels through rock, it is sometimes the practice to work the power drills by steam without the use of compressed air; in such cases, the steam is furnished from the boiler direct to the drill. In driving the Pryor Gap tunnel, on the Burlington & Missouri River Railroad, three power drills were used in each end, the drills taking the steam through hose from the boiler. A 16-horsepower upright boiler furnished sufficient steam to run the three drills. In one of the tunnels, previously referred to, fourteen power drills worked by compressed air were used at each end of the tunnel. All material taken into or out of the tunnel during construction was hauled by electric motors; the tunnel was lighted by electricity, and the foul air was drawn out by means of large centrifugal fans. In addition, a large quantity of water was pumped through the tunnel from the east end, which was at the upper end of the gradient. In order to furnish sufficient power to run all this machinery, a large power plant was installed at each portal of the tunnel. At the east portal, six 150-horsepower boilers with air compressors, pumps, dynamos, and fans were required; and at the west portal, three 150-horsepower boilers with the necessary machinery were used.

22. Ventilation.—When the tunnel heading is not very far from the mouth of the tunnel, and especially in clear weather, the gases formed by the combustion of the powder used in blasting will clear away with sufficient rapidity not to hinder the work; but in dull, foggy weather, and when the distance from the heading to the mouth of the tunnel is great, several hours may elapse after a blast before the tunnel heading will become even reasonably free from smoke. Contractors frequently make the serious mistake, from a false idea of economy, of neglecting to take the means necessary to clear the air so that men can properly work in it. Men cannot and will not do full work in a tunnel reeking with powder smoke. To do effective work, each man should have a supply of 100 cubic feet of pure air per minute. If there are twenty-five men in the heading, the supply of pure air should therefore be 2,500 cubic feet per minute. The 70-horsepower compressor that will be necessary to run the drill in such a heading will deliver to the drills about 500 cubic feet of free air per minute. So far as it goes, this is available for ventilation, but it only supplies one-fifth of the amount of air that is really necessary. The remainder may be supplied by blowers; but this method is objectionable on account of the fact that, when the pure air is forced into the tunnel by blowers, this means that the foul air must be displaced along the total length of the tunnel to the outlet. A preferable method is to use exhaust fans, which remove the powder smoke and foul air through pipes directly from the heading, and which by suction draw in a supply of fresh air from the outside of the tunnel.

23. Lighting.—The older method of lighting a tunnel during construction was by the employment of miners' lamps or of candles. These are exceedingly objectionable, because they not only consume the supply of fresh air, which in a tunnel has a considerable value, but they also add still more to the smoke of the tunnel and give out an offensive smell. Such a method of lighting has been almost entirely superseded by the use of electric lights, provided that a power plant is used in connection with the construction of the tunnel. Electricity has the advantage of furnishing a brilliant light, which consumes no oxygen and makes no smell or smoke. Almost its only disadvantage lies in the fragility of the globes and the danger of their being broken during the blasting.

24. Shafts.—A **tunnel shaft** is an opening or passageway extending from the surface of the ground to the grade of the tunnel. Shafts are usually made vertical and on the center line of the tunnel. They are used during construction to facilitate the work of excavating by affording access to the tunnel at intermediate points between the portals. Work is carried on in the tunnel in opposite directions from the bottom of a shaft; this shaft serves as a tunnel entrance through which excavated material is hoisted and surplus water pumped; and also as a passageway for pipes conveying compressed air and for admitting necessary construction material. Shafts are also useful in affording ventilation to adjacent sections of a tunnel and in facilitating ventilation through the tunnel after the excavation reaches the shaft from the portal.

25. Shaft Lining.—When a shaft is sunk through solid rock, the walls are self-sustaining and usually require no lining except a curb at the top of the shaft. When, however, a shaft is sunk through loose material, it is necessary to use a **lining** in order to prevent the sides from caving in. Such a lining is usually made of timber, and is put in, as the shaft is sunk, in the following manner: Rectangular wooden frames, shown in plan at *a*, Fig. 28, are made of the required dimensions to fit inside the shaft. These frames are spaced about 4 feet apart vertically as the shaft is sunk, and behind them is placed **lagging** on end, and in close contact, as shown at *b*. Such lagging may consist of sawed timber or of half-round poles split from young trees, as may be more convenient. As each frame is placed in position, it is supported by struts footing on the bottom of the shaft; or, if the walls are sufficiently firm, the frames are held in place by wedges, until another set is required, when timber struts *c*, mortised into the frames, form the permanent support. These struts are placed one above the other, and, together with the frames into which they are mortised, form continuous timber columns extending from the bottom to the top of the shaft. With each set of timbers, a horizontal timber *d*, called a **bunton**, is placed with ends abutting against the vertical timber *e*. A beveled seat with a square shoulder is cut on the vertical timber for each bunton. The buntons are held in place by wedges shown at *f*; these wedges are forced between the bunton and the shoulder of the beveled seat.

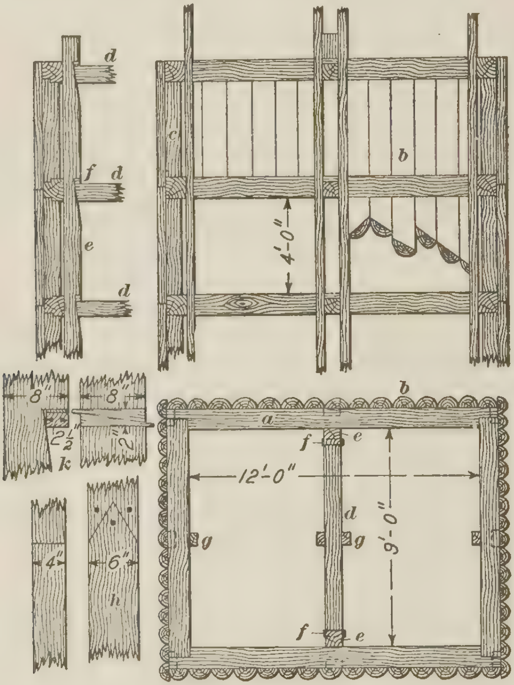

Fig. 28

As the wedges are tightened, the bunton is forced downwards until it is perfectly rigid. Vertical timbers *g*, *g* are spiked to the buntons and to the ends of the frames, to serve as guides for the carriage. A detail of the splice of the carriage guide is shown at *h*, and of the wedges at *k*. Since the pressure of the earth against the sides of the shaft lining is usually sufficient to hold the timbers in place, there will not be much necessity for using spikes or bolts for such purpose. Where fastenings of some character are required, it is better to use wooden pins in preference to iron, since most shafts are wet and iron will be liable to rust rapidly.

26. Removing Excavated Material.—The material excavated in tunnel driving is called **muck.** Various methods are employed for handling the muck, which is removed by loading it into dump cars at the foot of the bench and hauling it away. When there is a sufficient descending grade from the bench, the loaded cars can be run by gravity either to the foot of a shaft, where the material is hoisted to the surface, or to the waste dump outside the tunnel entrance. Where the grade will not permit running the cars by gravity, they are hauled by mules or by some form of steam or electric motor. The muck cars are operated on a track of 2 feet or 3 feet gauge, and are of capacities varying from 1 cubic yard to 3 cubic yards each. Usually, a double track is built, which allows loaded and empty cars to pass on separate tracks without delay. When the cars are hauled by mules, a single loaded car usually constitutes a load; but when a motor is used, the cars can be moved in trains of from ten to sixteen cars. When a single track is used, it is generally laid on the center line of the tunnel, with passing branches at suitable intervals.

In Fig. 29 is illustrated a good arrangement for handling the muck from a bench and heading over a single track, the same arrangement being suitable for a double track. At a distance of about 100 feet from the bench, a simple switch is built and two tracks *C* and *D* are laid to the bench, as shown. A plank runway *F* extends from the heading out over the tracks, being supported at suitable intervals on scaffolds *E*, *E*. The muck from the heading is loaded into barrows, wheeled on the runway a sufficient distance, and dumped directly into the cars. A simple and effective form of scaffold consists of two pieces of wrought-iron pipe, one telescoping within the other, both being provided with clamps by which they are adjusted to the required length. Such a scaffold is placed in position with the ends of the pipe abutting against the sides of the tunnel with sufficient force to support the runway and the load on it. The air for working the drills is carried from the air pipe *G* to the bench by means of the bench hose *H*. The manifold *K* attached to the end of the bench hose contains hose connections for all the drills. A ditch *L* on the opposite side of the tunnel from the air pipe drains the tunnel.

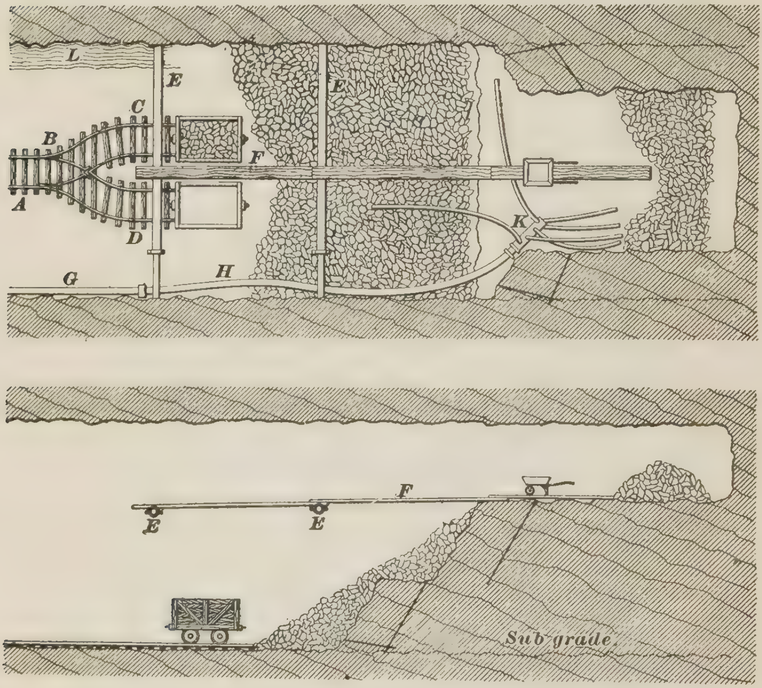

Fig. 29

27. Care of Track.—It is important to keep up a good track on which to haul the cars through the tunnel; and, in order to avoid frequent breakdowns and derailments, the track should be built of heavy material in a substantial manner. As the work advances from the tunnel entrance or from the foot of the shaft, the track will have to be lengthened to cover the increased distance. A common fault with contractors is a mistaken notion of economy in this particular. Worn-out and cracked rails, poor fastenings, poor ties or very few of them, although apparently cheap, are really expensive, since derailments are continually occurring, which involve costly delays. Short rails of varying lengths are required, so that the track may be kept continually within a short distance of the bench. A supply of rails and ties, together with the necessary track tools, should be constantly on hand, so that the track may be promptly added to as desired. When a single track is used, passing branches are built at suitable intervals, where returning empty cars are switched to pass the loaded cars going out.

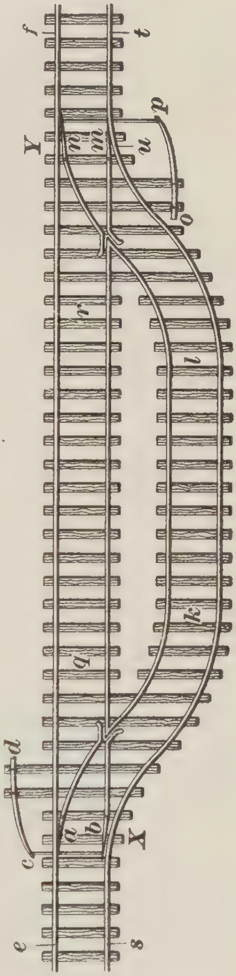

FIG. 30

A good form of passing branch is shown in Fig. 30; this is connected with the main track by two switches, which are made self-acting by the simple device shown in the figure. The points of the switch rails *a* and *b* are connected by a clamp rod, attached to a spring *c d*, which is constantly acting and holds the point *a* close against the main rail *e f*, and the switch is constantly set for the passing branch *k l*. The switch points *m* and *n* of the second switch are kept in place by the spring *o p*, the switch being always set for the main track *q r*. An outgoing car running in the direction *r q* finds the switch *Y* set for the main track. On reaching the switch *X*, the flange of the right-hand wheel, passing between the rail *e f* and the switch point *a*, forces the switch point *b* against the rail *s t*, and the car passes the switch in safety. A returning empty car finds the switch *X* set for the passing branch *k l*; in going from the branch to the main track, the flange of the head-wheel, as it passes between the rail *u t* and the switch point *m*, forces the switch point *n* against the main rail *e f*, and the car passes safely on to the main track. The springs *c d* and *o p* are elastic young saplings kept in place by strong staples driven into the switch ties.

28. Directing the Excavation.—It requires constant attention and watchfulness on the part of the engineers to direct the excavation in its proper direction and elevation. There is a constant tendency to keep above grade, which is possibly due to the unconscious effort to avoid the water that is constantly accumulating. Grade points should be established at least every 25 feet. Since the blasting cannot loosen the rock so that the excavated section will conform closely to the theoretical lines, the points of rock that project into the theoretical section after the blasted material is removed must be taken out by picks, wedges, and sledges. As this is slow and expensive work, it is usually a matter of economy to drill the blast holes so that they will reach a depth of about 1 foot beyond the limiting or theoretical section, so that the entire section will be removed by the blasting.

29. Timbering.—When the material is earth or rotten rock, the tunnel must be timbered. The timbered section should be enough larger than the standard section to admit of a masonry lining. When the material is such that the side walls will stand of themselves for a time, benches *A* and *B*, Fig. 31, are excavated near the springing line, and sets of timbers are placed as shown in the figure. Iron clamps, shown at *C* and *D*, hold the timbers together while the lagging *E*, *F* is being placed. The spaces *G*, *H* between the lagging and the roof are filled with dry rubble or cordwood.

Roof timbers should be either 12 in. × 12 in. or 12 in. × 14 in.; they should be placed with 2 to 4 feet clear space between two sets. Where the ground is very soft, with a tendency to expand, larger timbers may be necessary. Hemlock, yellow pine, or spruce is commonly used. In special cases, where great pressure is to be resisted, oak is used. When the side walls will not support the roof timbers, the latter are carried on supports arranged as shown in Fig. 32. Four posts *A*, *C*, *D*, and *B*, resting on sills *O*, *P*, *Q*, and *R*, are mortised into the cap *EF*. The roof timbers

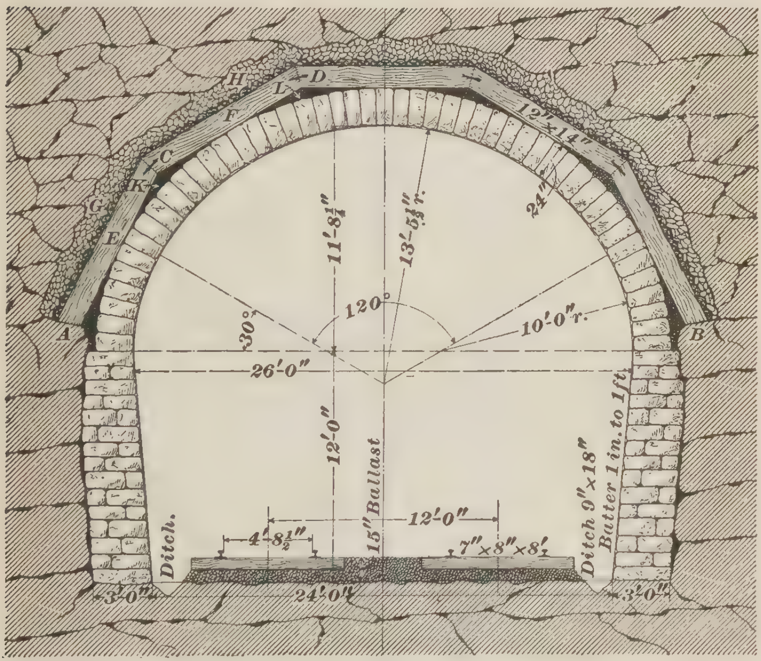

FIG. 31

G, *H*, *K* are clamped together as in Fig. 31, and mortised into the cap at the springing line. The pressure against the roof timbers is relieved by the struts *L*, *M*, *N*, *U*, and *V*, which transfer the pressure to the posts *C* and *D* and the cap *EF*. The dimensions of timber given in the drawing are such as are used where the pressure is great; they will meet the requirements of most situations. The lagging may be either sawed timber or split poles, obtained by splitting in half straight-grained chestnut or oak saplings. The backing may be either dry rubble or concrete. The side walls are all of well-scabbled rubble of good-sized stones, with even beds, laid in courses with cement mortar. The impost courses *S* and *T* should be of well-cut stone not less than 12 inches in thickness and the full width of the wall. The arch is of either brick or rubble. The caps and roof struts interfere somewhat with arching. Holes are left in the masonry where these timbers interfere until a section of the arch is completed, when they are removed and the gaps are filled with masonry, the joints being thoroughly grouted. All other timbers are left in place. The spaces *K*, *L*, etc., Fig. 31, between the arch and roof timbers are usually filled with concrete.

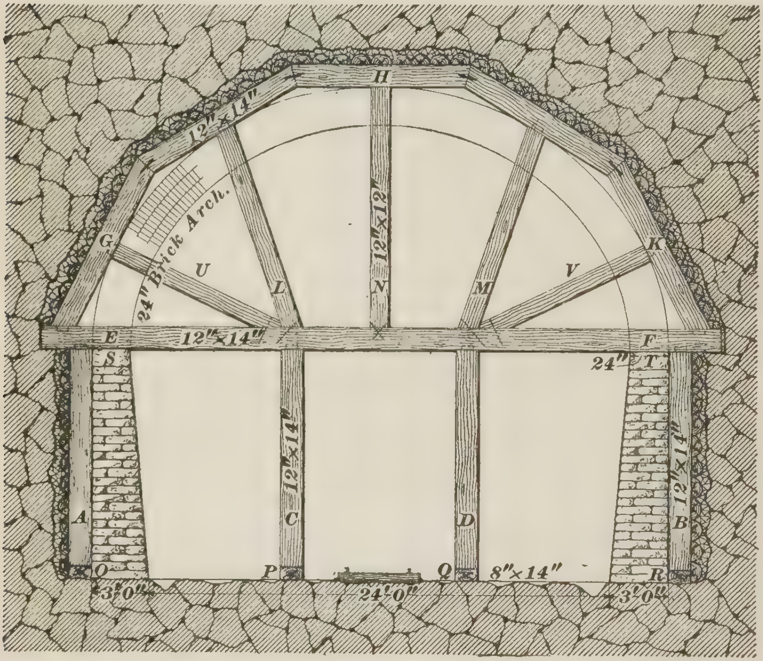

FIG. 32

30. When the material through which the tunnel passes is very soft, with slight coherence, all the energy and skill of engineer and workmen are required to make headway. It is considered the better practice to drive the heading at the bottom of the tunnel instead of the top, the reason being that, by the time the heading is driven, the ground composing the remainder of the section will have become thoroughly drained, and may be taken out with much greater safety and less expense than would be the case with a top heading. The mode of driving a heading through such material is illustrated in Fig. 33, which represents a cross-section and a longitudinal section of the heading, with a complete system of timbering.

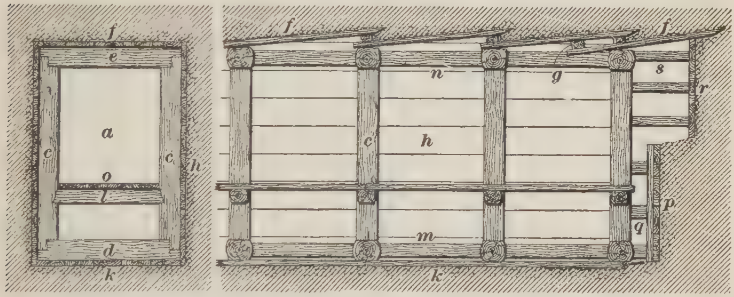

Fig. 33

A full section of timbers, as *c e c d* shown at *a*, is called a **set**; the upright timber *c* is called the **leg**; the horizontal timber *d*, the **sill**; and *e*, the **cap** or **collar.** The short boards *f*, *f* that extend from collar to collar and are in direct contact with the sustained material are called **poling boards.** They are sharpened to a cutting edge, and are driven into the face of the heading with sledges, a wedge-shaped block *g* being placed above them to keep them at a proper angle. The planks *h* that protect the sides of the heading are termed **lagging.** The flooring *k* serves to exclude the liquid mud, which would otherwise be forced from underneath by the external pressure. The horizontal cross-timber *l*, as well as the longitudinal timbers *m* and *n*, are called **struts.** The floor *o* serves as a footing for the workmen while driving the poling boards.

31. If the material penetrated is wet enough to run, it is necessary to constantly maintain a bulkhead of planks *p*, Fig. 33, called **face boards,** held in place by struts *q*. As the poling boards are driven forwards, the top face board is removed, allowing the released material to flow into the gangway. This forms a cavity in the face of the heading, and immediately another bulkhead is started by placing a face board *r* in advance of the boards at *p*, with a strut *s* to keep it in place. When the heading is advanced half the length of the poling boards, a new set of timbers is put in place, the collar of which takes the strain from the poling boards, which would otherwise be soon broken by the great pressure above them.

As the section is enlarged, other timbers are substituted, until the complete section is excavated. The masonry lining should follow immediately. The less important timbers may be removed as the masonry advances; but the main supports should remain in place; the masonry should be built around them, and they should not be disturbed until the arch is keyed. They can then be removed with safety, and the vacancies in the masonry carefully filled and grouted. All open spaces back of the masonry should be filled with well-rammed concrete.

32. Measuring Excavation.—Various methods are employed for determining the cross-section of the tunnel in order to compute the volume excavated. The best device is the following, illustrated in Fig. 34: A semicircular protractor *a b* with a diameter of from 8 to 10 feet, and made of light pine, is set up at right angles to the center line of the tunnel. The diameter *a b* of the protractor is brought into a horizontal position by means of the spirit level *c*, and placed at any desired height above the floor of the tunnel. It is not necessary that the center *d* should be on the center line of the tunnel, but its exact position with reference to that line should be very accurately determined. A sliding rod *d e*, one end of which is fastened to the center *d* of the protractor, measures the distances to the tunnel walls on radial lines. The angles that these lines make with the horizontal are read directly from the protractor. The tunnel section and the actual working measurements are then platted on cross-section paper, from which the area of the cross-section can be readily calculated, or it can be measured with a planimeter. Cross-sections are taken at intervals of about 10 feet. As the distance between them is so small, and as, besides, the areas are nearly equal, the volume between any two of them is computed with sufficient accuracy by the method of averaging end areas.

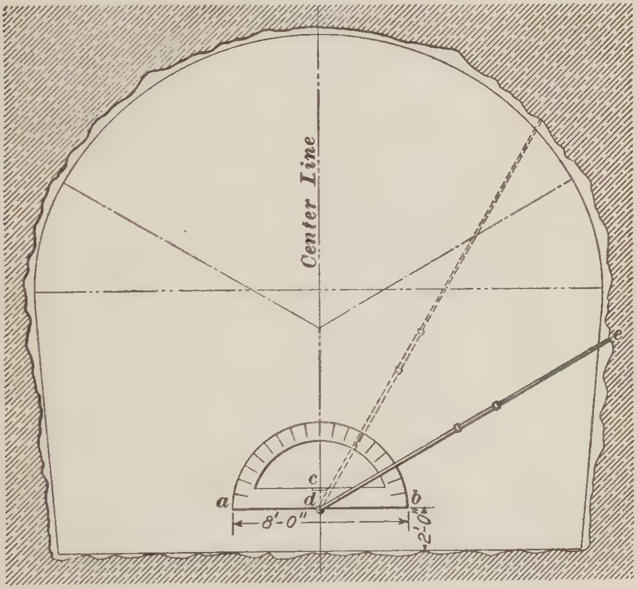

Fig. 34

TUNNEL LINING

33. Requirements.—Some kinds of rock are so hard and firm and so unaffected by exposure to the air that they may be safely left in their natural state without any kind of lining. Where lining is necessary, almost the only suitable material is masonry in some form. The character and thickness of the lining are purely matters of judgment and, unfortunately, of judgment that is little better than guesswork. Although a reasonably close estimate may be made of the capacity of a given form of lining to withstand a given stress, the amount and character of the stresses to which a tunnel lining will be subjected are frequently matters that can at best only be roughly approximated. If a tunnel is merely a subsurface tunnel carrying a definite load on the surface, the problem is comparatively simple; but when, as sometimes occurs, the tunnel passes through strata of soft material, which are themselves in a state of unstable equilibrium, there is a tendency to distortion and crushing by forces that cannot be accurately known or measured. The only method to be followed in such cases is to adopt a lining that is probably strong enough, and to watch the tunnel carefully for any evidence of settlement or failure.

34. Timber and Masonry Lining.—Timber is frequently used as the so-called permanent lining of a tunnel, but it hardly deserves the name of permanent, since it cannot be expected to last indefinitely. If the outside of the tunnel were perfectly dry, as is the case with the framing timbers in the roof of a house, or if, on the contrary, the timbers were constantly wet, as they would be under water, they might be expected to last almost indefinitely; but, in a tunnel, neither of these conditions obtains, the timber being subject to constant alternations of wetness and dryness, which promote rapid decay. A timber lining should therefore be constructed with the idea that it will be renewed or replaced by a masonry lining. Sometimes, the tunnel is excavated with such a width that, even after the timber lining is put in, there is sufficient space to build a masonry lining covering the timber and still have sufficient clearance room for the cars. This, however, is bad practice, especially when the ground is soft and treacherous, for in time the timber will decay utterly and permit the earth above the tunnel to drop down on the masonry lining. The impact of a great mass of soil is likely to develop a high pressure that may cause serious damage to or even crush the masonry lining. A better way to substitute a masonry for a timber lining is to take out all the timber, even the cordwood that is frequently used to fill up the irregularities caused by the blasting, and to substitute a masonry lining with a backing of concrete to fill up all gaps.

35. Arch Masonry Lining.—When the earth pressure becomes very great, it may be necessary to put in a masonry lining that is arched, not only at the top, but also on the sides and bottom. In Fig. 35 is shown an illustration of the lining of an Austrian tunnel. The side walls were constructed in an arched form of brick, which rests on a line of stone quoins at the lower corners. The dotted lines across the bottom show how, in especially soft ground, even an inverted arch was considered necessary. The timbering is very heavy. Two headings were made, both in the center line, one at the bottom and the other at the top of the final full section. It should be noted that the brickwork is carried behind the arch line proper, so as to entirely fill the space between the arch and the lagging, the timbers being entirely taken out as fast as the brick lining was sufficiently complete to allow the safe removal of the timber.

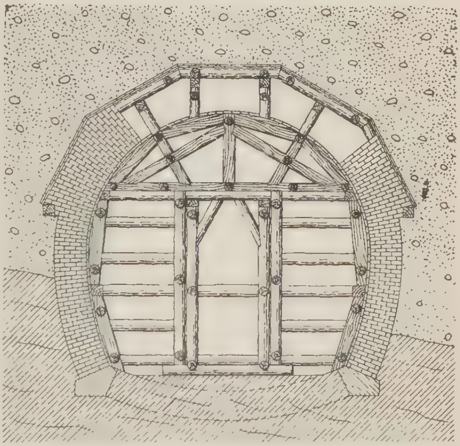
FIG. 35

36. Centers and Molds.—In putting in a masonry lining for a tunnel, forms or patterns are used to show the exact outline of the masonry required. Such patterns vary in number and shape according to the form of lining and the kind of masonry used. Where the excavation is lined on the bottom, as well as on the sides and roof, the masonry work is divided into three parts; namely, the invert, or floor, masonry, the side-wall masonry, and the roof-arch masonry. A separate pattern is required in constructing each part of the lining; these patterns are called, respectively, *ground molds*, *leading frames*, and *centers*. The shape and construction of each form or pattern will be described.

37. Ground Molds.—A **ground mold** for tunnel masonry consists of a wooden frame or pattern of exactly the form and dimensions of the cross-section of the masonry used for the floor lining. A good form of ground mold is made of plank about 3 inches thick; it is of such shape that its lower edge fits the surface of the excavation, and its upper edge fits the curve of the masonry floor. Such a mold is illustrated in Fig. 36. In order to afford convenience in handling them in a restricted space, ground molds are usually made in two parts, which are joined together in the middle by an iron plate on each side, as shown in the figure. The plates are bolted through from side to side. In using the molds, they are placed at right angles to the axis of the tunnel if the tunnel is straight, and on radial lines if the tunnel is on a curve. They are centered by means of a transit, and leveled until the top of the mold is at exactly the proper elevation. Usually, two molds are employed, at a convenient distance apart, and cords are stretched tightly from one to the other to mark the outline of the masonry. Sometimes, only one mold is employed, in which case the outline of the finished masonry is used instead of another mold.

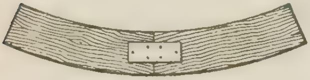

FIG. 36

38. Leading Frames.—As already stated, the forms or patterns for the side walls are called **leading frames.** They are usually made of plank of suitable thickness; one edge is cut to the curve of the excavation of the side of the tunnel, and the other edge is made to conform to the face of the side wall. In Fig. 37 is shown a form of leading frame suitable for use in a tunnel whose side walls slope inwards toward the bottom. Leading frames, when in position for use, are commonly set with the lower portion in contact with some fixed part of the ground. The position of the top can be fixed by means of a plumb-line or spirit level, as may be convenient. In Fig. 37 is shown the relative position of a ground mold and a leading frame when set for masonry.

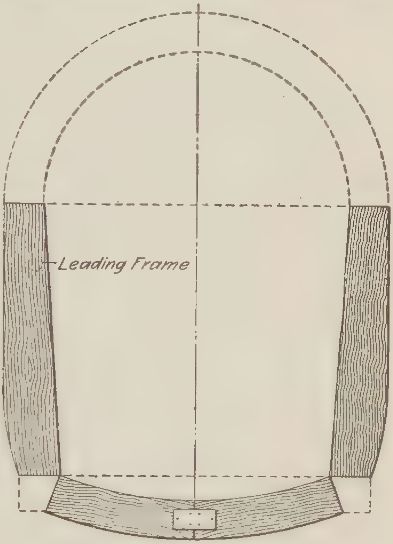

FIG. 37

39. Centers.—The form on which the roof arch masonry of the tunnel is built is called a **center.** Tunnel centers are usually required to support the roof pressure, in addition to the weight of the masonry during the construction of the roof arch, and for this reason they must be made strong. They are generally similar in form to centers used for ordinary arch masonry. In tunnel work, owing to the confined space in which the work must be done, it is customary to omit the center or radial bracing of the arch centers. Where heavy loads have to be supported, the tunnel centers are trussed in such a manner as to leave room for working under the central position. A good form of trussed center is shown in Fig. 38. Where the lining is made of concrete, a special form of center is used, which is mounted on wheels and is moved on a track through the tunnel as the work progresses.

40. Lagging.—The **lagging** consists of narrow strips secured to the outer surface of the arch frames, and spanning the opening between adjacent centers. The outer surface of the lagging is made to conform exactly to the curve of the roof arch, and forms the mold for the surface of the arch masonry. For stone masonry, the strips of lagging are placed so as to come under the joints between the ring stones, leaving an interval between adjacent strips. For brick and concrete arches, the strips are laid close together. The lagging should be strong enough to hold up the arch masonry, and also to support the short props that keep the poling boards in place after the timber supports have been removed, and until the masonry is completed.

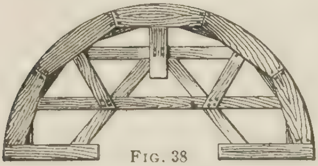

FIG. 38

41. Striking the Centers. The centers are brought to the proper elevation by means of wooden wedges inserted between the bottom of the posts that support the centers and the floor of the tunnel. These wedges are usually made of hard wood, and are about 6 inches wide, 4 inches thick, and 18 inches long. After the masonry is completed, these wedges are withdrawn, allowing the center to drop clear of the masonry; this operation is called **striking the centers.** Usually, the centers are not struck immediately after the masonry is completed, but are allowed to remain a sufficient time to allow the mortar to set hard.

42. Portals.—The name **portal** is applied to the facing that is usually built at each end of a tunnel. Even when the main body of the tunnel is not lined, it is usual to construct a facing of masonry at the end, as the roof of the tunnel is usually rather thin at the ends, and the disintegrating action of the weather on a roof of solid rock might, in time, cause a dangerous breakage. It sometimes happens that the natural surface outside the end of the tunnel is nearly or entirely vertical. When this occurs, the material is usually very solid rock, and it may be safe to leave the portal unlined. If the soil over the mouth of the tunnel is composed entirely or mainly of earth, it is not only necessary to have a lined portal, but it requires unusual care to construct the portal without inducing landslides. Structurally, the portal has very much the same appearance as the end of a large arched culvert. Above the arch is the usual parapet, from the top of which the earth slopes back, the slope being such as is consistent with stability. The height of the parapet above the arch depends on the depth to which the open-cut approach is made before the tunnel is begun. From each side of the tunnel two wing walls are run out. If the open-cut approach to the tunnel is very long, and is unusually deep at the portal end, the wing walls may run parallel to the line of the road, with a gradually diminishing height. In other cases, side walls are run out from the portal face perpendicular to the line of the road.

The construction of these portals is a serious problem for the engineer. Sometimes it is effected by sinking a shaft at such a distance from the portal that the natural roof of the tunnel is so deep and strong that the tunnel section between the shaft and the portal may be excavated and lined without difficulty. The excavation then proceeds back toward the portal, and by the time the very thin roof immediately over the portal is reached, the lining of the tunnel prevents any serious landslide, and it is comparatively easy to break through. If the portal is constructed by beginning at the outside, there is a danger that the loosening of the earth during the tunnel excavation will cause a landslide that will sweep away the tunnel timbering. This is sometimes prevented by anchoring the timbering immediately back of the portal with a mass of heavy stones, so that, if a landslide occurs, it will not disturb this weighted timbering.

Frequently, the design of tunnel portals is made somewhat elaborate, but this must be considered a matter of architecture rather than of engineering.

DAMS

INTRODUCTION

1. Classification of Dams.—Dams are divided into several classes, according to the material of which they are composed. The principal classes, and those that will be described in this Section, are **masonry dams,** which are built of masonry, either cut stone, rubble, or concrete, or a combination of these materials; and **earth dams,** which are built of earth, with a masonry center wall. Some dams are built of earth alone, but unless the dam is of a very insignificant height, this kind of construction is not to be recommended.

When rock on which the proposed dam can be built is found on the bottom and at the sides of the valley, a masonry dam is always to be preferred. If no rock is found, so that the dam must be built on an earth foundation, an earthen dam, with a masonry center wall, is greatly to be preferred, as being more secure. If, as is frequently the case, rock exists on one side of the valley and not on the other, the dam may be of a composite character; the center wall and earth embankment must be placed on the side where there is no rock foundation.

2. Preparations for Construction.—After a comparison of all the locations examined and studied, both on the ground and on paper, the site of the dam is definitely determined. The next thing is to prepare for construction. For this purpose, the center line of the dam is run out and established by permanent monuments, either of stone, or,

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although less advantageously, of strong posts driven into the ground with their heads nearly level with the surface, so that an instrument can be conveniently set over them. Two of such monuments should be set at each end of the center line, perhaps 100 feet apart, making four in all, so that there shall be no danger of losing the line by the destruction or displacement of any two of them. The whole area that is to be in any way worked over in the construction of the dam is then cross-sectioned by dividing it into squares 25 feet or less on a side, and taking the elevations of all the corners, with intermediate points if the topography of the ground requires it. The squares with the elevations of their corners above datum are mapped in duplicate or even triplicate. One copy should be kept in a place of perfect security, as this map constitutes a record of the ground before any work was commenced, and from it all the excavation and embankment—in a word, all the work that changes the face of the ground—are calculated for the partial and final estimates. The importance of taking and securing this record must not be underrated, as it is the only way to know what work has actually been performed, and to settle disputes that may arise regarding such work. This cross-sectioning will require a great number of stakes to be driven. As these stakes are not permanently preserved, they may be of very light material.

At least two permanent bench marks are established in convenient locations, and, by means of these, temporary ones can be set as they are needed for the work.

EARTHEN DAMS WITH MASONRY CENTER WALLS

CENTER WALL

3. **Purpose.**—In earthen dams, the **center wall is the** first thing to be considered. This wall should be carried down to a water-tight foundation, if possible, and its ends should be deeply embedded in the sides of the valley. Its object is two-fold: first, to afford an impermeable and indestructible cut-off to any water that might otherwise percolate through the bank, either because the bank itself is not impermeable, or because it has been perforated by muskrats or other burrowing animals; second, to afford a means of making water-tight connections for the culverts or pipes used for conveying water from the reservoir. Without such water-tight connection as the cut-off wall offers, there is always danger that the water may flow outside of these pipes, form a channel in the bank, and finally cause the destruction of the dam. Several instances of failures from this cause are on record.

4. **Foundations.**—In building the center wall, the first consideration is the foundation. The wall should be carried down until a stratum is encountered that appears to be impermeable. Generally speaking, the finer the material, the better it is as a foundation. Fine gravel or sand is probably the best; but even when these are mixed with clay, they form an excellent material on which to found the center wall. Fine sand with smooth, round grains, mixed with a large percentage of clay, constitutes what is known as **quicksand,** and when found at a certain depth below the surface, so that it cannot escape laterally, forms a good water-tight foundation. Loose, coarse gravel containing

large stones or cobbles is about the worst material that can be encountered. When such material is found, the center wall must be carried down to a much greater depth than would otherwise be necessary. Increased depth of foundation compensates to a considerable extent for poor quality of material.

5. **Dimensions.**—The center wall should be carried up as high as the level of the highest water in the reservoir. Its thickness at the surface of the ground should be one-quarter of the height, and this thickness may be reduced by a batter of 1 inch to the foot on both sides, or, preferably, by building the wall with vertical sides and stepping in 2 feet (1 foot on each side) every 10 feet, which amounts to about the same thing. As the foundation ascends the sides of the valley, it is stepped up, but care must be taken to keep the bottom of the wall well below the surface of the ground and well embedded in good material. The ends of the wall must also be carried well into the banks forming the sides of the valley.

6. **Puddle Walls.**—Sometimes, the masonry center wall is replaced by a puddle wall. Puddle is a mixture of clay and gravel or sand, which are well mixed together, carefully moistened, and rammed in place. When well made of selected materials and properly placed, it makes a safe and satisfactory wall, and is somewhat cheaper than masonry. This class of construction is general in England, where the work is thoroughly understood, and carried to a high degree of perfection. It is very much less common in the United States, where the masonry center wall has become typical of American practice. Since the puddle wall requires very careful selection of materials and many minute precautions in their preparation and placing, the work must be performed by skilled labor. For these reasons, a masonry wall is generally to be preferred.

7. **Concrete Walls.**—Concrete can be advantageously employed for the construction of the center wall. A center wall built of this material is very strong, because of its

monolithic character; and, as the concrete work is done by machinery and unskilled labor, it is cheaper than a masonry wall.

EMBANKMENT

8. **Form and Material.**—The center wall, as described, forms the **core** about which the **earthen embankment** is formed. This embankment rises to a certain height above the top of the wall, which height depends on circumstances to be considered hereafter. The embankment is flat on the top, and has a gentle slope on each side, the rate of slope depending greatly on the material of which the bank is formed. The best material is a fine gravel or coarse sand, such as is suitable for making mortar. Clay, although perfectly water-tight when confined, is a treacherous material in a bank, because it is so fine that it actually dissolves in the water, and is liable to be washed away in the form of semifluid mud. Its cohesiveness is also a very objectionable feature; for, if a cavity is formed in a clay bank, it remains open indefinitely, whereas the tendency of a bank composed of sand and gravel is to close all such openings. Thus, if a bar is forced into a mass of clay and then withdrawn, it leaves a round hole, the size of the bar; but if the same thing is done in a bank of fine gravel or sand, the space occupied by the bar is immediately filled up by the material, when the bar is withdrawn.

9. **Method of Building.**—The earthen embankment should be carried up in horizontal layers, and kept constantly moist by sprinkling. In many specifications, it is provided that these layers must be consolidated with a roller. This is of doubtful expediency, for the tendency of rolling is to form separate and distinct layers or "partings" in the bank. If the material is brought to the bank in wagons, and evenly spread by shovels and horse scrapers, being kept constantly moist meanwhile, the travel of the men, horses, wagons, and scrapers is generally all that is needed to secure a good bank.

Before placing the embankment, the natural surface on

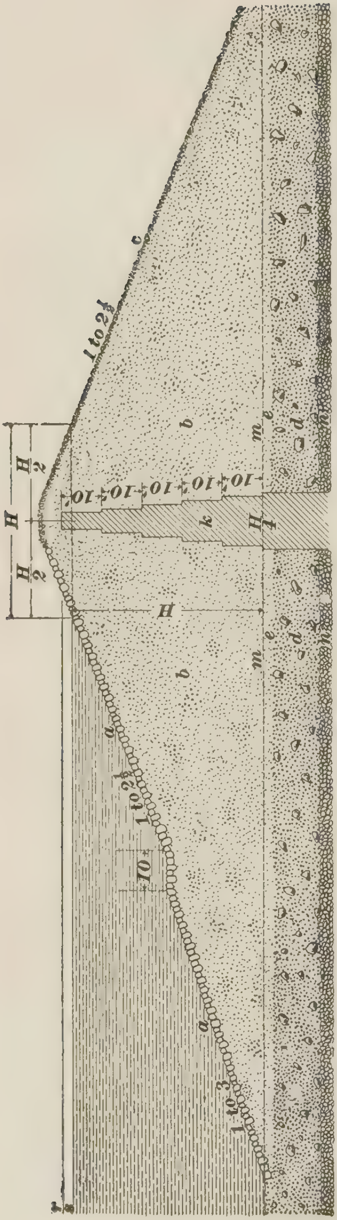

FIG. 1

which it is to stand must be carefully stripped of all sods, roots, and vegetation, so that the first layer of the earthen embankment may rest on and incorporate itself with clean earth. This is particularly necessary under the inner slope.

10. Slope.—A fair average for the outside slope, that is, the slope on the lower or down-stream side *c*, Fig. 1, is 1 vertical to 2½ horizontal. A somewhat flatter slope is advisable on the inside or water side *a* of the bank, say 1 vertical to from 2½ to 3½ horizontal. In a high embankment, say 50 feet or more, it is better to divide the inside slope into two or more steps, as shown in Fig. 1. From the top, a slope of about 1 to 2½ is carried down for 25 or 30 feet, and then a *berm* 8 to 10 feet wide is introduced. From this point, the slope is continued with a somewhat flatter grade, as 1 to 3, for another 25 or 30 feet, when another berm is introduced, and the slope is again continued, either with the same grade as before or, preferably, with a grade somewhat flatter, as 1 to 3½.

The inside slope, next to the water, should be carefully paved, or ripraped with stone for a thickness of from 1 foot to 2 feet, from the bottom to a point well above the high-water line. The greatest thickness should be between the first berm, as above mentioned, and the top of the dam, for that part is most exposed to wave action. The stone composing this paving must be placed and packed by hand, not dumped at random. The outside slope should be sodded, or at least sown with grass and carefully tended until a good sod is formed.

11. Designs of Dams.—Fig. 1 shows all the constructive features in a general way, and represents a good type of earthen dam. In this figure, *a* is the inner slope of the embankment; *b*, *b*, the earthwork embankment; *c*, the outer slope; and *d*, *d*, the natural loose material under the embankment from which the vegetable soil has been stripped, as shown by the lighter shading at *e*, *e*. The original surface of the ground is shown by the dotted line *m*, *m*; *k* is the center wall, carried well down into the compact sand and gravel *n*, *n*. The line *r* shows the high-water level, or level of freshet overflow, and *s* is the low-water level, or level at which overflow begins. It will be perceived that several of the dimensions depend on H, which is the vertical distance from the natural surface of the ground to the level of the overflow, or lip of the spillway. If the dam crosses a deep and wide stream, H must be taken equal to the vertical distance from the bottom of the stream to the lip of the spillway.

The thickness of the earth embankment at the level of the lip is taken equal to H. The earth embankment is evenly divided by the center line of the dam. The center wall is given a thickness at the base equal to $\frac{H}{4}$, and at every 10 feet it is stepped in 1 foot on each side. There is also a small offset, or footing, given to the foundation, which is carried down to a secure formation of fine sand and gravel. The top of the center wall is carried up to the height of high-water level in the reservoir, or to a height equal to $H + D$, D being the depth of notch of the spillway.

The following illustrative example will show how the above principles are applied:

12. Referring to Fig. 1, let $H = 48$ feet, and $D = 2.5$ feet. The center wall commences with a thickness of 12 feet. At a height of 10 feet, it is drawn in by offsets to 10 feet; at 20 feet, to 8 feet; at 30 feet, to 6 feet; and at 40 feet, to 4 feet, which thickness is carried through to the top. The total height of the center wall above the foundation is $48 + 2.5 = 50.5$ feet. The embankment is carried up 8 feet above the level of the spillway; at the level of the spillway, or low-water line, it has a thickness of 48 feet, the top width being 8 feet. At a depth of 27 feet below the spillway, a berm 10 feet wide is introduced, and the slope is continued at the rate of 1 to 3. These dimensions and proportions represent safe minimum values under ordinary conditions; they will be found to furnish a sound basis for the designs to which they apply, subject to modification for local conditions.

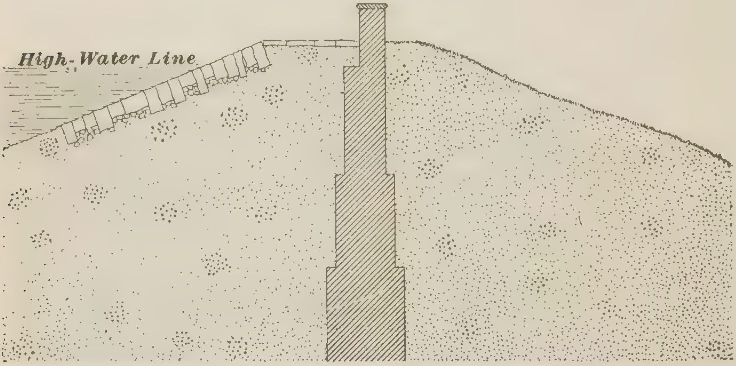

FIG. 2

13. Another Design.—An excellent, though somewhat more expensive, design for the top of the dam is shown in Fig. 2. In this form of dam, the center wall is extended through the embankment and forms a parapet wall along the top. If the top of the bank is flagged, as shown in the figure, the work is very complete and secure. The top of the bank should slope away from the wall on both sides, to prevent water from working down between the bank and the wall. The possibility of such an occurrence is perhaps an objectionable feature of this style of construction, which otherwise has much to recommend it.

SPILLWAY, OR OVERFLOW

14. The **spillway** is one of the most important features of a dam. It is the means by which the surplus water, when the reservoir is full, is allowed to run to waste, and want of sufficient discharging capacity in this particular has been probably the most common cause of destruction of earthen dams. If such dams are once overtopped by a flood, especially when not provided with a proper center wall, they are rapidly cut down and destroyed by the water running over them.

Frequently, a natural overflow can be found in some lateral depression of the ground, by which the surplus water can be passed into another valley. When such an overflow occurs, particularly if the depression is in rock, it is generally used, since, besides being conducive to safety, it saves a considerable expense. Sometimes, it is possible to form such an overflow by cutting into some rocky ridge leading either into another valley or into the same valley, lower down, across which the dam is built. In the majority of cases, however, it is found necessary to provide a special piece of masonry construction for a spillway. The spillway is then called a **waste weir.**

15. Dimensions.—The dimensions of a spillway must be proportioned to the amount of water likely to go over in times of freshet. It may be comparatively long and shallow, or short and deep. A convenient length is given by the formula

$$L = 20\sqrt{A}$$

in which L = length, in feet, of lip of spillway;
A = area, in square miles, of watershed above the dam.

Having determined the length, the depth is computed by the weir formulas given in *Hydraulics*. In applying these formulas, a careful determination must be made of the amount of water to be discharged.

16. Quantity of Water to be Provided For.—If dams are already built in the stream under consideration, or in neighboring streams of nearly the same volume, an examination of their overflows, or spillways, will afford valuable information in designing the new work. If there is a bridge under which all the water of the stream passes, some conception of the amount of flood discharge to be provided for may be obtained by a knowledge of the greatest height to which the water has been observed to rise on the abutments. Otherwise, the discharge, or run-off, can be approximately computed by the Buerkli-Ziegler formula, given in *Sewerage.*

In sections of the country where heavy ice is formed in winter, the depth of the spillway should be sufficient to float the thickest ice that may be formed in the reservoir; otherwise, should a freshet occur when the reservoir is filled with ice, the spillway may be blocked with the broken cakes carried down by the wind and current.

17. Time as a Dimension Factor.—It is clear that the duration of a storm exercises a marked influence on the height to which the water will rise in the reservoir, and consequently on the depth D of the head of water going over the spillway. The time t, in seconds, required for the water to rise to a certain height H_1 over the lip of the spillway, while at the same time discharging over it, may be determined approximately by the following empirical formula, which applies when H_1 is from two-thirds to three-fourths of the total height D to which the given flood may rise:

$$t = \frac{A_1 H_1}{Q_1 - 2 \times L \times H_1 \sqrt{H_1}}$$

in which A_1 = average area of the water surface in the reservoir, in square feet;

L = length of spillway, in feet;

Q_1 = cubic feet of run-off entering the reservoir per second.

Example.—Assume the area of the water surface in a reservoir when full to be 10,000,000 square feet. If the length of the spillway is 155 feet and the discharge is 7,320 cubic feet per second, how long will it take the water to rise to a height of 4 feet?

Solution.—Here, A_1 = 10,000,000, H_1 = 4, Q_1 = 7,320, and L = 155; substituting these values in the formula,

$$t = \frac{10{,}000{,}000 \times 4}{7{,}320 - 2 \times 155 \times 4\sqrt{4}} = 8{,}264 \text{ sec., or 2 hr. 18 min., nearly. Ans.}$$

EXAMPLES FOR PRACTICE

1. The approximate area of a reservoir is 25,000,000 square feet. If the length of the spillway is 150 feet, and the discharge 7,500 cubic feet per second, how long will it take the water to rise to a height of 2 feet in the spillway? Ans. 2 hr. 2 min.

2. The approximate area of a reservoir is 10,000,000 square feet. If the length of the spillway is 90 feet and the discharge 6,800 cubic feet per second, how long will it take the water to rise to a height of 3 feet in the spillway? Ans. 1 hr. 25 min.

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18. Artificial Spillway.—When no natural overflow, as already mentioned, is available, an artificial one must be built. This will generally be located in the line of the stream across which the dam is erected, although the spillway may sometimes be placed at or near one end of the dam, if the character of the ground and other conditions favor this construction. If rock is not found on which to build the structure, the foundations must be carried well down, perhaps deeper than those of the adjacent center wall.

19. Cross-Section of Spillway.—The form and dimensions of the cross-section of the spillway vary greatly; excellent examples from actual structures show considerable difference in the ideas of their designers. There are no fixed rules governing this point. One of the principal differences that will be noticed in existing works is the form of the face of the spillway over which the water passes. This is sometimes formed in steps, and sometimes in a concave curve. When the latter form is used, the face stones should be cut, at least roughly, to voussoir shape, and their faces, over which the water passes, dressed smooth. In the best work of this class, the facing is of cut stone throughout. This is, of course, very expensive. The object sought in using steps is to break the force of the falling water, so that it will reach the bottom with very little velocity, either vertical or horizontal. The idea embraced in the curved face is exactly the contrary, the object sought—or at least attained—being the greatest possible horizontal velocity urging the water forwards away from the foot of the spillway when it has reached the bottom. Of the two, the step system seems preferable, as the water appears to reach the bottom with less destructive force when its fall is thus broken. Of course, lower down in the stream, the velocity of the water is the same, whichever system is adopted. For low dams, say up to 15 or 20 feet, the face may be nearly vertical, giving the water a clear fall on the apron at the bottom. For higher dams, say up to 60 or 70 feet, the form shown in Fig. 3 is a very good one.

Let AB represent the height from the bottom of the spillway to the level of high water, or freshet flow, in the reservoir. Take AD equal to nine-tenths of AB; also, take AC equal to AD. Join C and D. The face EF has a batter of 1 in 12. Since F, the intersection of EF and CD, is the point where the steps begin, it is the essential point on a working drawing. As the slope of CD is 1 in 1, KF, the horizontal distance of F from AC, is equal to CK, the vertical distance of F below C. But the face batter of GF is 1 in 12; therefore, F is one-twelfth of CK farther from AC than G is; that is,

$$KF = CG + \frac{CK}{12}$$

or, since $CK = KF$,

$$KF = CG + \frac{KF}{12}$$

The top width CE is made about one-fifth of AC. The face CE is given a batter of about 1 in 4 or 5, and is made of cut stone. The corner E is rounded off.

Spillway sections for still higher dams constitute special cases and can better be considered after a study of high masonry dams.

EXAMPLE.—Referring to Fig. 3, let $AB = 70$ feet, and let the vertical height of high water over E, the lip of the spillway, be 3.5 feet. To determine the other dimensions.

SOLUTION.—In Fig. 3, $AD = \frac{9}{10} \times 70 = 63$ ft. $CB = 70 - 63 = 7$ ft. Since E is 3.5 ft. below high-water level, it is $7 - 3.5 = 3.5$ ft. higher than C. Now, CE is to be about $\frac{1}{5} \times 63 = 12.6$ ft. But this dimension admits of a good deal of latitude, and if CG is made 13 ft., CE, though less than 12.6 ft., will be long enough. Hence, CG is taken as 13 ft. Then, $x = CK = KF = \frac{12}{11} \times 13 = 14.2$ ft. This fixes the point F; and from this point, the face EF is drawn with a batter of 1 in 12, up to within 3.5 ft. of high-water level. Ans.

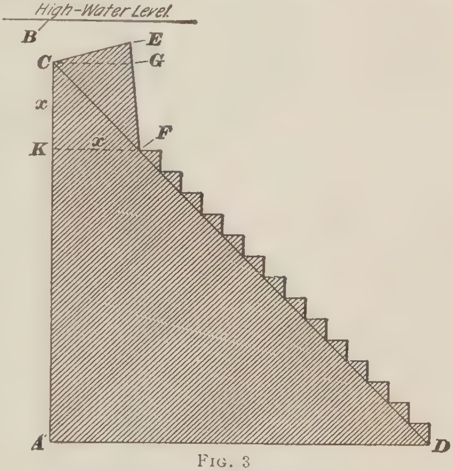

FIG. 3

20. Accessories of Spillway.—The sides of the spillway where it cuts through the embankment must necessarily be protected by wing walls to prevent the earth from falling into it. These wing walls need not have as flat a slope as the exterior embankment, as this would make them unnecessarily large and expensive. They may have a slope of about 1 to $1\frac{1}{2}$, with the bank graded down to them. The top of the wall may have a coping, well doweled into the stones, or be left with the horizontal courses forming steps; the latter form of construction, though not so neat in appearance, is more substantial. When the bed of the stream is not rocky, the foot of the spillway must be protected by an apron composed of very heavy stones, if they can be obtained, laid in cement on a bed of concrete. This apron must be extended well

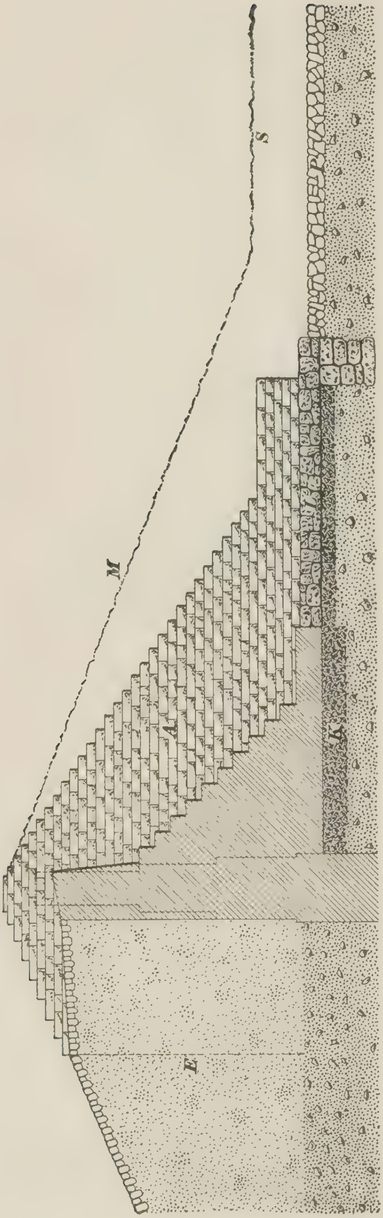

FIG. 4

beyond the foot of the spillway, the distance depending on the height of the dam, the volume of water passing over it, and the nature of the bed of the stream. It should never extend less than a distance equal to the height of the dam. Beyond this, it will be well to protect the bed of the stream between the banks for some distance farther with heavy, dry stone paving. All these features are shown in Fig. 4. No dimensions are given in this sketch, because the conditions of each particular case will greatly modify the details; but the general arrangement and proportions will always resemble those shown. In this sketch, the wing wall of ashlar masonry is shown at *A*; *M* is the exterior slope of the embankment, and *S* the bank of the stream below the dam. The apron *L* is composed of heavy stones, laid in cement mortar on a bed *K* of concrete. There is a cut-off at the end of the apron, which may advantageously be built of concrete instead of rubble masonry, as shown in the figure. The dry stone paving *P*, composed of heavy stones, forms a continuation of the apron.

APPLIANCES FOR DRAWING OFF THE WATER

21. Requisites.—There are many devices for the control of the water. Their main requirements are that they be simple and effective and not liable to get out of order. In all cases, a communication must be established between the inside and the outside of the dam, and it is of the utmost importance that no water shall follow along the outside of the appliance—whether tunnel, gallery, or pipe lines—by means of which this communication is established. Herein lies one of the great advantages of a substantial center wall, for it affords the means of making a water-tight connection in the center of the embankment, beyond which no water can trickle. As these features of the dam call for a considerable amount of heavy and expensive masonry, it is also important to design them so that the necessary degree of solidity may be secured with the least possible volume of masonry. This is best accomplished by grouping the different parts together, so that they may be mutually supporting, and so that a portion of one may also form a portion of another.

A system of valves is indispensable for the control of the water. In small reservoirs, these valves may be inserted in the pipe line outside the reservoir and have only an ordinary valve box to indicate their location. Generally, however, they are at the reservoir end of the line, the valve stem thus being below water level, so that a shaft or a tunnel becomes necessary to secure access to the valves.

22. Details of Tunnel Method.—The **tunnel method** is common in European practice, and involves a masonry core wall and a tunnel from the lower face through the embankment, the pipe line being laid in this tunnel. The draw-off pipes are laid through the core wall and are carried down through the tunnel open to view and available for repairs.

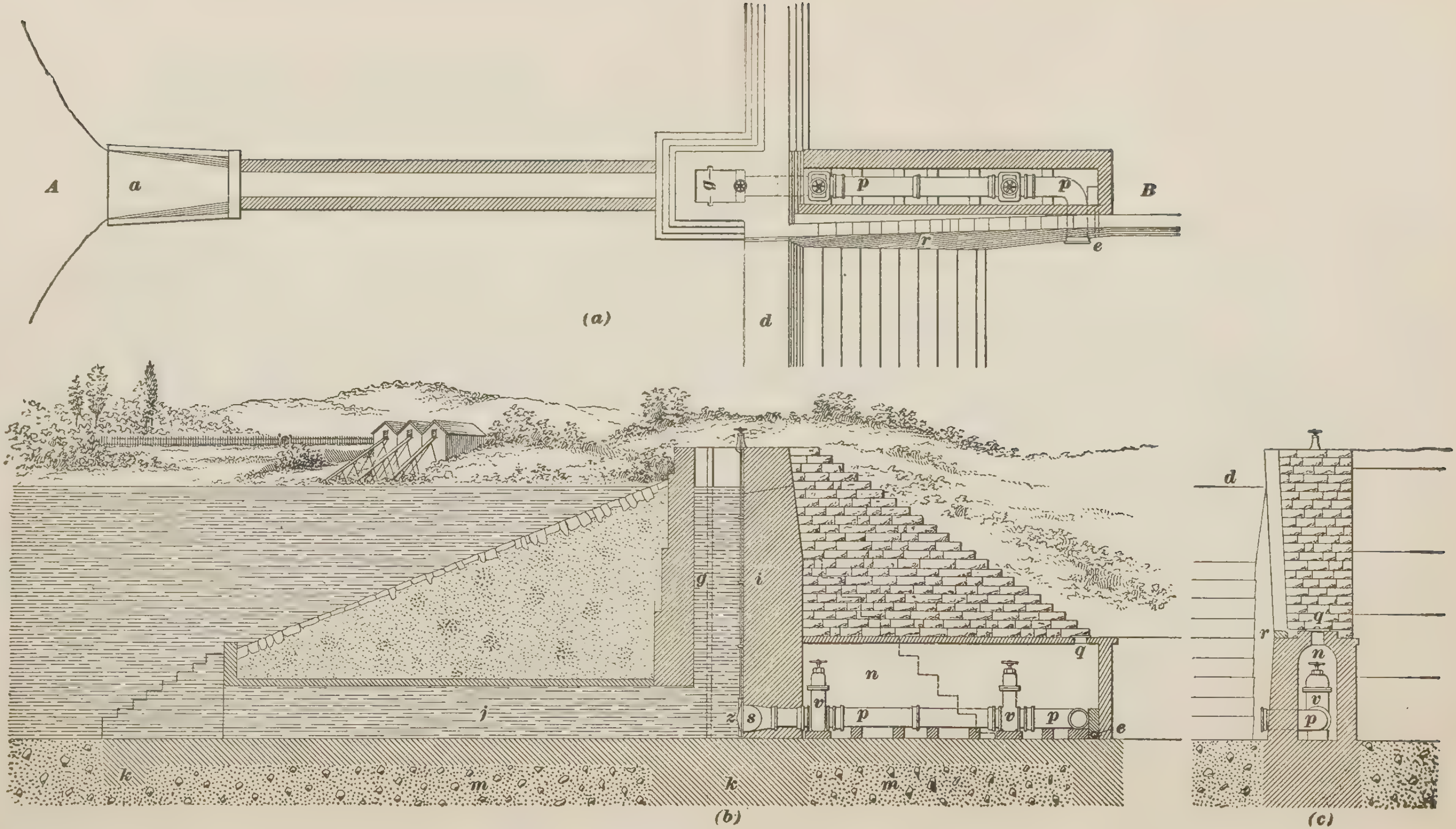

FIG 5

Fig. 5 (*a*) shows a plan and (*b*) a vertical section on the line *AB* of a general system of design that experience proves to be strong and satisfactory. In the same figure, (*c*) shows a front elevation and section through the gatehouse *n*, looking toward the spillway from outside the reservoir, it being assumed that the embankment is removed. On the inside of the dam, adjacent to the spillway *d*, is built a water tower *i*, one side of which is formed by the prolongation of the spillway itself. This tower communicates with the inside of the reservoir by means of an arched gallery or tunnel *j*, which passes under the interior embankment and terminates in an open portal *a* with wing walls. The top of the tower is level with the top of the embankment. The reducer *s* leading to the cast-iron pipe *p* is securely built into the opposite wall of the tower, and the pipe itself is placed in an arched gallery *n* under the exterior embankment; one wall of this gallery forms part of the main wing wall *r* of the spillway. The reducer *s* is a special casting having a rectangular opening at the face, the area of which is equal to or greater than the area of the pipe to which the other end is fitted. The pipe *p* discharges, in some convenient way, into the channel of the stream below the dam. In this way, a clear communication is established between the inside and the outside of the reservoir.

The water passing through this system can be controlled in several ways. The best way is to close the mouth of the reducer on the inside of the tower with a sliding sluice gate *z*, as shown in the figure. Besides this, there should be a stop-cock, or valve, on the pipe inside of the exterior gallery *n*, by which the letting on or shutting off of the water is ordinarily effected. The sluice gate *z* is kept open, and is closed only in case of emergency, such as an accident to the valve. If no such sluice gate is provided, there should be two stop-cocks on the pipe. In the figures, two valves *v*, *v* are shown in addition to the sluice gate, but generally this is not considered necessary.

Whichever of the two systems is employed—and the single-valve and sluice gate is probably the preferable one—there should always be a set of grooves *g* cut in the masonry of the tower, in which, in an emergency, stop-planks may be placed. **Stop-planks** are heavy timbers of suitable dimensions that are slipped one by one into the grooves; being reeinforced with iron plates, these planks sink readily in the water. The iron plates may be made advantageously in the form of angle irons, so that by means of hooks the plank can be withdrawn. In case it is desired to gain access to the mouth of the pipe without emptying the reservoir, these stop-planks constitute a very valuable means of shutting off the water. Access to the gallery *n* is provided by the manhole *q*, and the gallery is kept free from water by the small drain *e*. The whole structure rests on a hydraulic rubble or concrete foundation *k*, *k*, which must extend down to rock or solid ground *m*, *m*.

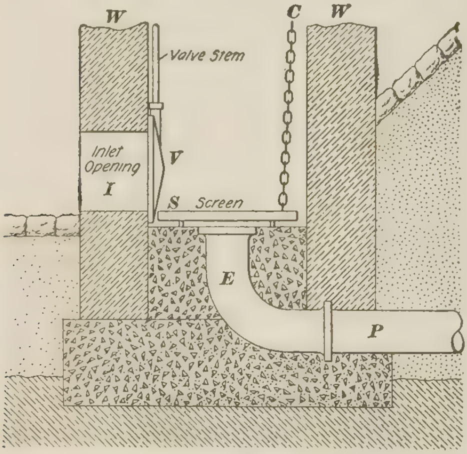

FIG. 6

23. Details of Shaft Method.—In the **shaft method,** the outlet pipe is laid from a point outside of the reservoir on the lower level, through or around the embankment, to a tower or shaft to which the water has direct access. This tower may be either entirely away from the dam, or at the inside toe of the embankment slope; or it may be built shaft-like through the center of the dam, water being brought in by one or more pipes. Fig. 6 shows the simplest construction at the toe. The pipe *P* is brought in under the embankment, and turned up vertically by an elbow *E* ending under a screen *S*, which, to be cleaned, may be lifted by means of the chain *C*. Water enters the tower *W*, *W*, which may be circular or rectangular, through the inlet opening *I*, and flows out through *E* and *P*. The inlet *I* is provided with a valve *V* operated from the floor above by a long valve stem.

Fig. 7 shows a more elaborate design, on the same general plan as that just described. The tower is built, as before, at the inside toe of the embankment, and is connected at the top by a light footbridge; P_1 and P_2 are the outlet pipes, and g, g_1, g_2 are three inlets through the masonry; *S*, *S* are a series of rod screens through which the water has to pass; P_3 is the drain pipe to be used occasionally, when all the water is to be drawn off. Valves, not shown, are provided for the pipes P_1 and P_2 and at the inlet openings g, g_1, g_2.

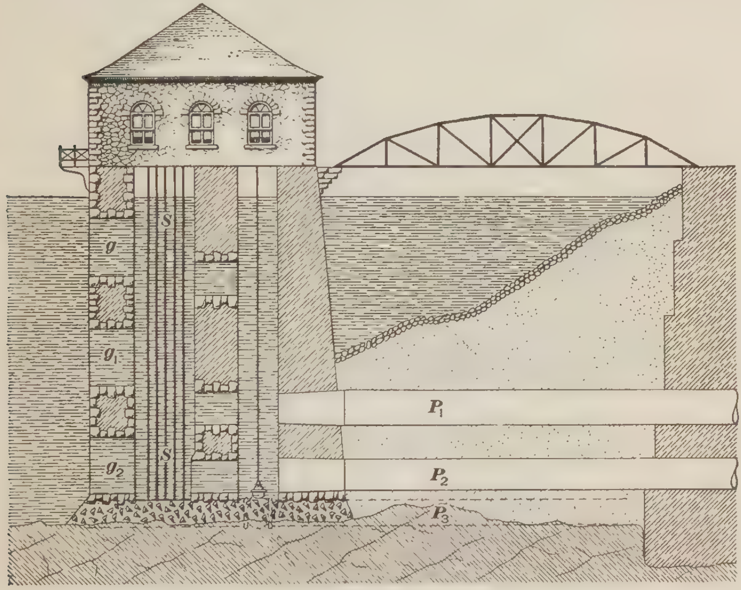

FIG. 7

Fig. 8 shows the simplest type of the method in which the tower is placed in or near the center of the embankment: *W*, *W* are the walls of the tower; g, g_1, the inlet openings at different levels; *P*, the outlet pipes; and the triangle *C*, two concrete or masonry walls, between which the water has

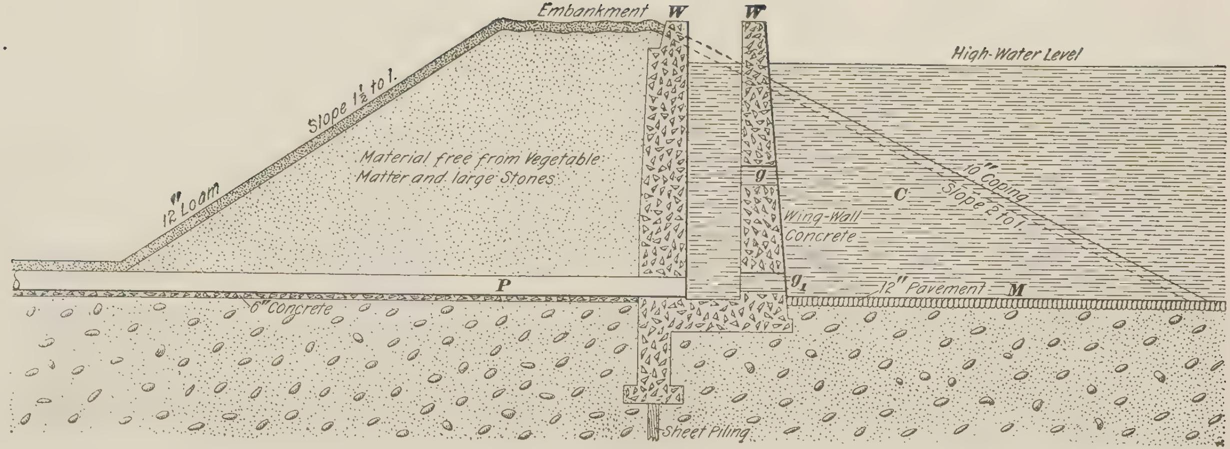
Embankment
W
W
High-Water Level
Slope 1½ to 1.
12" Loam
Material free from Vegetable Matter and large Stones
g
C
10" Coping
Slope 2 to 1.
Wing-Wall Concrete
g₁
12" Pavement
M
P
6" Concrete
Sheet Piling

FIG. 8

access to the tower. A pavement M is also provided to prevent washing.

Fig. 9 modifies this last design by substituting for the entrance walls a single entrance pipe, the exit pipe P_2 remaining as before. The figure shows the valve V_1 on the main pipe and the valve V_2 on the drain pipe D. The pipe P_1 has its end protected by a small wall W.

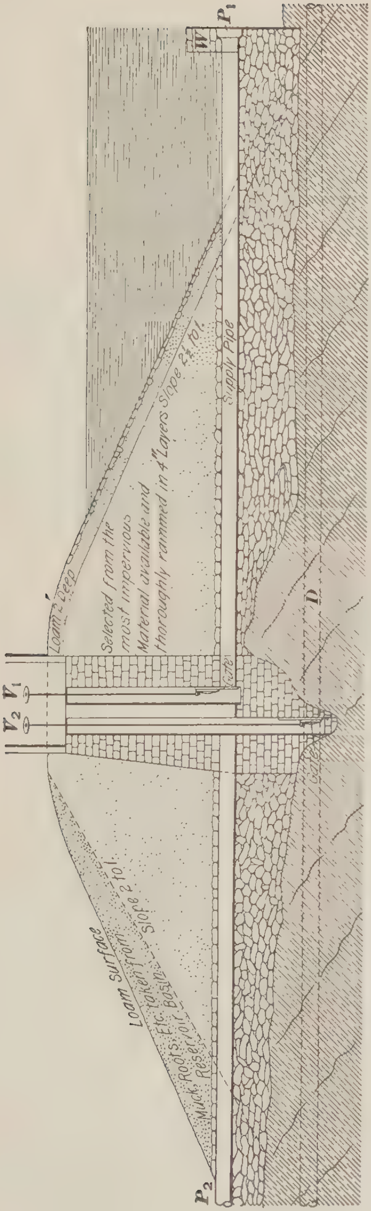
P1
W
Supply Pipe
Selected from the most impervious Material available and thoroughly rammed in 4" Layers Slope 2½ to 1.
Loam 2' Deep
Gate
D
V1
V2
Loam surface
Muck, Roots, Etc. taken from Reservoir Basin
Slope 2 to 1.
P2

FIG. 9

24. Modifications for Small Reservoirs. For small reservoirs, the appliances and arrangements described above are frequently simplified. For instance, the tower, the object of which is to afford means of applying the sluice gates and stop-plank, may be dispensed with, and the pipe run through the center wall to the foot of the interior slope; or, the arched gallery may be continued to the center wall and then connected with the pipe. In these cases, the only control of the water will be by means of the stopcocks in the exterior gate house.

MASONRY DAMS

LOW MASONRY DAMS

25. General Considerations.—In the structures hitherto treated, there has been no attempt made to determine their dimensions by calculation, as they do not lend themselves to exact mathematical treatment. In earthen dams, all dimensions are fixed by empirical rules; that is, experience has taught that a certain thickness of bank and certain ratios of slope lead to safe results. In masonry dams, the case is different. Given a wall sustaining a certain head of water, it is easy to calculate almost exactly the character, intensity, point of application, and direction of action of the destructive force or forces bearing on it. The resistance of the wall can also be computed with a great degree of approximation. The task of designing such structures is, therefore, more satisfactory than in the case of earthen embankments, and admits of a more scientific course of procedure.

26. Thrust Against Dam.—Expressed in pounds, the horizontal component T of the thrust against any surface $A B$, Fig. 10—whether vertical as at (a), inclined as at (b), or curved as at (c)—is the same, and is equal to half the square of the height H or head of water, in feet, pressing against $A B$, multiplied by 62.5, which is the weight, in pounds, of a cubic foot of water, and by the length of the surface. The horizontal thrust T_1 per foot of length of the dam is, therefore, given by the formula

$$T_1 = \frac{62.5\,H^2 \times 1}{2} = 31.25\,H^2 \qquad (1)$$

Also, the point of application of this thrust is the same for (a), (b), and (c); namely, at one-third of the height H

from the bottom of the wall. Hence, for the overturning moment M_1 of the horizontal thrust about the point C, we have

$$M_1 = 31.25\,H^2 \times \frac{H}{3} = 10.42\,H^3 \qquad (2)$$

In this formula, M_1, expressed in foot-pounds, is the moment about C of the thrust acting on 1 foot of length of the dam. Although the real resulting pressure acting on the dam is normal to the surface of the latter, and usually has a vertical component, this component is generally small, and may be neglected. The error arising from neglecting this component is on the side of safety.

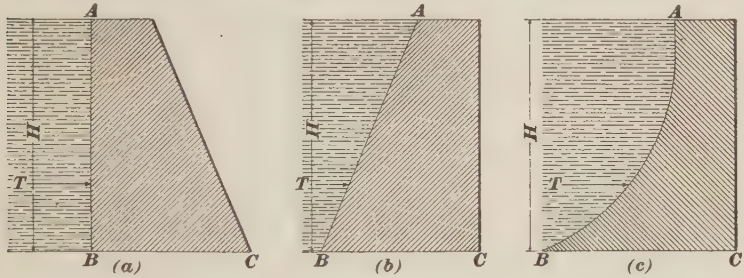

FIG. 10

EXAMPLE.—The depth H of water pressing against the curved surface $A\,B$, Fig. 10 (c), is 23 feet 7 inches. (a) What is the intensity, in pounds, of the horizontal thrust per foot of length of wall? (b) What is the overturning moment, in foot-pounds, about the point C, per foot of length of wall?

SOLUTION.— (a) $T_1 = 31.25(\frac{283}{12})^2 = 17{,}380$ lb. Ans.
(b) $M_1 = 10.42(\frac{283}{12})^3 = 136{,}680$ ft.-lb. Ans.

27. Tendency of the Thrust.—The thrust T tends to move the wall in two ways; namely, it tends to push the wall bodily and make it slide horizontally along its base; and it tends to upset the wall by making it turn about the point C, Fig. 10. The first of these tendencies must be resisted by the friction between the dam and its foundation; the second, by the weight of the dam. The action of these resistances will now be explained.

28. Resistance to Sliding.—The frictional resistance that the dam opposes to the tendency of the thrust of the water to move it forwards on its base is equal to the weight of the dam multiplied by the coefficient of friction between the dam and the material on which the dam rests. Much uncertainty attends the determination of the coefficient of friction; an ordinary estimate places it at about .75. It must be noted that friction only is now considered, as if the dam were merely standing on a level base with no mortar joint intervening. The adherence of the mortar and the bond of the work are both neglected. The resistance thus estimated is, therefore, much below the real resistance. It is at least safe, however, and will here be adopted. As will be presently shown, the tendency of the wall or dam to slide forwards is not, under ordinary circumstances, the destructive force most to be feared.

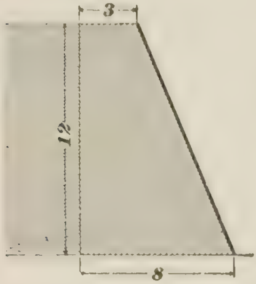

FIG. 11

In all calculations relating to the resistance of such walls as are now being considered, a length of 1 foot, or a slice of the wall 1 foot thick, is always taken, because, since the whole of the wall, if of uniform height, is made up of a succession of such slices, what is true of one will be true of all. The convenience of this consists in the fact that the area of a vertical cross-section of the wall is then numerically equal to the volume of the slice in cubic feet. Throughout all the following calculations, the units of length and weight are, respectively, the foot and the pound avoirdupois.

If R is the resistance of the wall, per foot of length, to sliding, and W is the weight of the wall per foot of length, then

$$R = .75\,W \qquad (1)$$

If the vertical cross-section of the wall is denoted by A, and the weight of the material per cubic foot is denoted by w, formula **1** may be written

$$R = .75\,A\,w \qquad (2)$$

EXAMPLE 1.—A trapezoidal wall, Fig. 11, 12 feet high, 3 feet wide at the top and 8 feet at the bottom, weighs 115 pounds per cubic foot. To determine: (a) its resistance to sliding; (b) its factor of safety.

Solution.—(*a*) Substituting in formula **2**,

$$R = .75 \times \frac{8+3}{2} \times 12 \times 115 = 5{,}692.5 \text{ lb. Ans.}$$

(*b*) In order to determine the factor of safety, it is necessary to find the amount of thrust. From formula **1** of Art. **26**,

$$T_1 = 31.25 \times 12^2 = 4{,}500 \text{ lb.}$$

Then, the factor of safety is equal to the resistance divided by the thrust, or $5{,}692.5 \div 4{,}500 = 1.27$. Ans.

Example 2.—Let the wall in example 1 be built of granite weighing 170 pounds per cubic foot. Required: (*a*) its resistance to sliding; (*b*) the factor of safety.

Solution.—(*a*) Substituting in formula **2**,

$$R = .75 \times \frac{8+3}{2} \times 12 \times 170 = 8{,}415 \text{ lb. Ans.}$$

(*b*) The factor of safety is equal to the resistance divided by the thrust, which was determined above. Then, the factor of safety is equal to $8.415 \div 4{,}500 = 1.87$. Ans.

29. Resistance to Overturning.—The resistance to overturning, called the **moment of stability** of the wall, is the static moment of the wall with reference to the point about which rotation tends to take place, that is, about the toe *C*, Fig. 12, of the dam; it is equal to the weight of the wall multiplied by the horizontal distance between the toe and a vertical line through the center of gravity of the wall. In order that there may be equilibrium, the moment of stability of the dam per foot of length must be equal to the moment M_1 of the horizontal thrust, as found in Art. **26**.

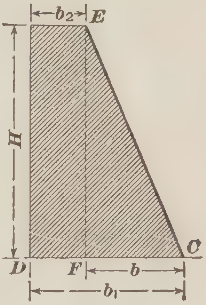

Fig. 12

The almost invariable section or profile of a masonry dam, except for very high dams, is a trapezoid, as shown in Fig. 12. The section may be divided into a rectangle *DE*, whose area is $b_2 H$, and a triangle *EFC*, whose area is $\frac{1}{2}(b_1 - b_2)H$.

The static moment of the rectangle about the toe *C* is

$$b_2 H\left(b + \frac{FD}{2}\right) = b_2 H\left(b_1 - b_2 + \frac{b_2}{2}\right) = b_2 H\left(b_1 - \frac{b_2}{2}\right)$$

The static moment of the triangle about the toe C is, since the center of gravity is horizontally distant $\frac{2b}{3}$ from C,

$$\tfrac{1}{2}(b_1 - b_2)H \times \tfrac{2}{3}(b_1 - b_2) = \tfrac{1}{3}H(b_1 - b_2)^2$$

For the static moment M_2 of the whole area, we have, then,

$$\begin{aligned} M_a &= b_2 H\left(b_1 - \frac{b_2}{2}\right) + \tfrac{1}{3}H(b_1 - b_2)^2 \\ &= \frac{H}{3}\left(3\,b_1\,b_2 - \frac{3\,b_2^2}{2} + b_1^2 - 2\,b_1\,b_2 + b_2^2\right) \\ &= \frac{H}{3}\left(b_1^2 + b_1\,b_2 - \frac{b_2^2}{2}\right) \end{aligned}$$

The moment of stability M_s of the dam, per foot of length, is $w\,M_a$, denoting by w the weight of the material per cubic foot. Replacing M_a by its value, we have,

$$M_s = \frac{w\,H}{3}\left(b_1^2 + b_1\,b_2 - \frac{b_2^2}{2}\right) \qquad (1)$$

In order that the dam may be secure, M_s must be at least equal to M_1; hence, the least moment of stability that is consistent with equilibrium is given by the formula (see Art. 26) $\frac{w\,H}{3}\left(b_1^2 + b_1\,b_2 - \frac{b_2^2}{2}\right) = \frac{31.25\,H^3}{3}$;

whence,

$$H = \sqrt{\frac{\left(b_1^2 + b_1\,b_2 - \frac{b_2^2}{2}\right)w}{31.25}} \qquad (2)$$

Example.—What is the moment of stability of the trapezoidal wall of example 1, Art. 28?

Solution.—Here $b_1 = 8$, $b_2 = 3$, $w = 115$, and $H = 12$. Substituting in formula 1,

$$M_s = \frac{115 \times 12}{3}\left(8^2 + 8 \times 3 - \frac{3^2}{2}\right) = 38{,}410 \text{ ft.-lb.} \quad \text{Ans.}$$

30. Design of the Profile.—In designing masonry dams, the height H is determined by the amount of water to be stored, taken in connection with the nature of the place in which the reservoir is built; the weight w is given by the material of which the dam is built; the top width b_2 and the factor of safety are assumed. From these data, the width of base b_1 can be calculated.

Consider first the resistance to sliding. If a factor of safety j is used, formula **1** of Art. **26**, for the horizontal thrust, becomes $T_1 = 31.25\,j\,H^2$.

The resistance to sliding, from formula **2** of Art. **28**, is, writing $H\left(\frac{b_1 + b_2}{2}\right)$ for the area,

$$R = .75\,H\left(\frac{b_1 + b_2}{2}\right)w.$$

Since R must equal T_1, we must have

$$31.25\,j\,H^2 = .75\,H\left(\frac{b_1 + b_2}{2}\right)w;$$

whence,
$$b_1 = \frac{83.33\,j\,H}{w} - b_2.$$

Example 1.—The height of a trapezoidal wall is 30 feet; width at top, 6 feet; weight per cubic foot, 140 pounds; and factor of safety, 2.5. To determine the required bottom width of the wall to resist sliding.

Solution.—Here, $j = 2.5$, $H = 30$, $w = 140$, and $b_2 = 6$; substituting these values in the last equation,

$$b_1 = \frac{83.33 \times 2.5 \times 30}{140} - 6 = 38.64 \text{ ft.} \quad \text{Ans.}$$

31. To determine the breadth of base of a trapezoidal wall to resist overturning, the moment of stability given by formula **1** of Art. **29** must be equal to the moment of thrust given by formula **2** of Art. **26**. Introducing the factor of safety j, formula **2** of Art. **26** becomes

$$M_1 = \frac{31.25}{3}\,j\,H^3$$

Making the moment of thrust equal to the moment of stability,

$$\frac{31.25}{3}\,j\,H^3 = \frac{w\,H}{3}\left(b_1^2 + b_1\,b_2 - \frac{b_2^2}{2}\right)$$

Solving for b_1,

$$b_1 = \frac{1}{2}\sqrt{\frac{125\,j\,H^2}{w} + 3\,b_2^2} - \frac{b_2}{2}$$

Example.—The dimensions of a trapezoidal wall and the factor of safety being the same as those given in the example of the preceding article, determine the width of base to resist overturning.

Solution.—Substituting the given values in the formula,

$$b_1 = \frac{1}{2}\sqrt{\frac{125 \times 2.5 \times 30^2}{140} + 3 \times 6^2} - \frac{6}{2} = 20 \text{ ft.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. The depth of water pressing against a dam is 75 feet. (*a*) What is the horizontal thrust per foot of length of dam? (*b*) What is the overturning moment, in foot-pounds, about the outer toe of the dam, per foot of length of dam? Ans. $\begin{cases} (a) \ 175{,}780 \text{ lb.} \\ (b) \ 4{,}395{,}900 \text{ ft.-lb.} \end{cases}$

2. A trapezoidal wall 25 feet high is 4 feet wide on top, 12 feet at bottom, and weighs 130 pounds per cubic foot. What is its resistance to sliding? Ans. 19,500 lb.

3. What is the moment of stability of the wall in example 2? Ans. 199,330 ft.-lb.

4. The height of a trapezoidal wall is 50 feet; width at top, 8 feet; weight per cubic foot, 170 pounds; and factor of safety, 2.5. What is the required bottom width to resist: (*a*) sliding? (*b*) overturning? Ans. $\begin{cases} (a) \ 53.3 \text{ ft.} \\ (b) \ 30.6 \text{ ft.} \end{cases}$

---

32. Remarks on Stability.—In examining the results given by the formulas of Arts. **30** and **31**, it is evident that a much greater width of base is required to insure security against sliding than is required to guard against overturning. It must be kept in mind, however, that, as already stated, only the mere friction of stone on stone is taken into account when calculating the resistance to sliding; that is, only such friction as might occur if two level surfaces of stone were brought in contact. When it is remembered that a well-bound piece of masonry is by no means in this condition, but is knit together in a more or less homogeneous mass, it will be seen that the tendency to move forwards is counteracted, not by mere friction alone, but also by the resistance to shearing of the stonework. This is a very strong combination, and makes the total resistance so great that experience proves that, when a dam is safe against overturning, it is safe against being moved forwards bodily on its base. If, however, the whole dam were placed on a smooth surface, such as a timber grillage, particularly if the planks were laid in the same direction as the pressure, or if it rested on yielding clay, with only a small depth of foundation, very serious doubts might exist as to whether it would remain immovable, and careful examinations and calculations would be necessary. In such cases, the coefficient of friction may fall considerably below .75. As, however, all masonry dams should stand on a rock foundation, into which the footing course is well embedded, no danger of their moving bodily forwards need be apprehended if the stability is satisfactory as regards overturning.

33. Average Dimensions.—Calculations made with various practical values for w and b_2 show that a bottom width equal to from $\frac{2}{3}H$ to $\frac{3}{4}H$ will always give a satisfactory factor of safety, and in nearly all cases the smaller of these two values, that is, $b_1 = \frac{2H}{3}$, will give a perfectly secure profile.

HIGH MASONRY DAMS

34. General Considerations.—The trapezoidal profile hitherto considered is the one almost universally adopted for masonry dams up to 50 or 60 feet in height. Beyond this limit, it would no longer be economical nor, in very high dams, practicable. So far, only resistance to sliding and overturning has been considered, but in very high dams another element of destruction must be taken into account; namely, the crushing of the material under its own weight. In the case of symmetrical figures, the amount of pressure per square unit of base is obtained by dividing the whole

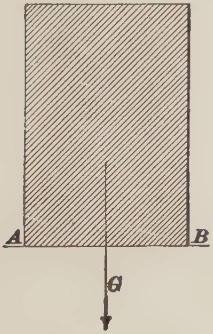

FIG. 13

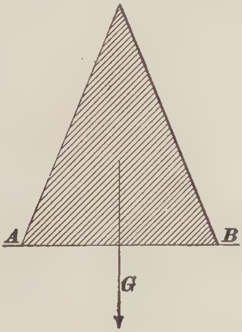

FIG. 14

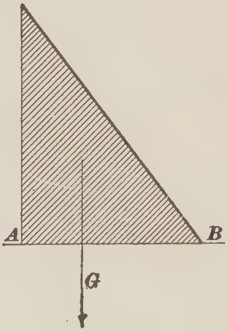

FIG. 15

weight resting on the base by the number of square units in the base. Thus, in Figs. 13 and 14, if W represents the total weight of the mass above the base AB, the uniform weight borne per square unit of the base is $\frac{W}{AB}$. If, however, the profile is that shown in Fig. 15, since the figure is not symmetrical about the line G passing through its center of gravity, the weight is not uniformly distributed over each square unit of base, and it is impossible, merely at sight, to say what the maximum intensity of stress may be. All that is clearly evident is that the intensity of stress must be greater on the shorter of the two segments into which the line of action G of the weight cuts the base.

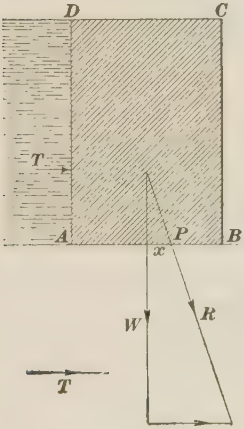

Fig. 16

Even when the profile is symmetrical, the action of an external force may cause the resultant pressure on the base to move from its central position toward one extremity of the base. Thus, in Fig. 16, $ABCD$ represents the profile of a rectangular wall sustaining water pressure; the total pressure R on the base is the resultant of the weight W of the structure and the thrust T of the water. The line of action of R does not intersect the base at the center, but at a point P whose distance x from the center can be readily determined by the principles of statics.

35. Unequally Distributed Load.—Suppose AB,

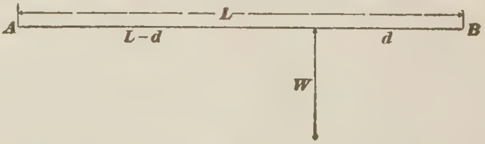

Fig. 17

Fig. 17, to be the base, or any given horizontal course, of a mass of homogeneous masonry. Let W be the vertical component of the resultant of its weight and any exterior force acting on it; let the line of action of this component divide the length L into two segments d and $L - d$, as shown. The maximum intensity of stress is in the shorter segment d, and its amount may be determined by the following formulas, which are derived by the use of advanced mathematics:

$$P = \frac{4W}{L^2}(L - 1.5d) \qquad (1)$$

$$P = \frac{2W}{3d} \qquad (2)$$

In these formulas, P is the maximum intensity of stress, or maximum pressure per unit of area, on the surface AB. Formula **1** is to be used when d is equal to or greater than $\frac{L}{3}$, and formula **2** when d is equal to or less than $\frac{L}{3}$. When $d = \frac{L}{3}$, the two formulas give the same results. Formula **1** is, with a slight change of notation, given in *Foundations*, Part 1.

Good authorities advise that formula **2** be used with great judgment, and only when d is but little less than $\frac{L}{3}$. Others consider the results obtained by formula **2** to be too small when the obliquity of the resultant of the weight of the mass and the thrust of the water exceeds a certain amount, the error increasing with the obliquity. A preferable formula, though one that has not yet been generally accepted, perhaps because it is not well known, is the following:

$$P = \frac{W(L - d)}{Ld} \qquad (3)$$

This formula applies to all values of d, and will be used throughout this Section. It gives very nearly the same results as formula **1** for cases in which the latter applies. The three formulas agree for $d = \frac{L}{3}$. For values of d less than $\frac{L}{3}$, formula **3** gives pressures increasingly greater than formula **2**, which, to conform to the ideas of some good authorities, it should do. It will be explained later, however, that in designing the profile of a masonry dam, whether high or low, the point where the resultant pressure cuts the base should always be kept within the "middle third" of the base, and as near the center as possible. That is, d should always lie between $\frac{L}{3}$ and $\frac{L}{2}$, and the nearer the latter, the better.

36. Conditions of Stress.—In the study of the stresses sustained by a high masonry dam, two conditions are to be considered: the stress when the dam is supporting only its own weight, and the stress when the dam is under the action of the water pressure, in addition to its own weight. These two conditions correspond, respectively, to an empty and a full reservoir. In the former case, the resultant pressure is between the center of the base and the inside toe of the dam; in the latter case, the resultant pressure is between the center and the outside toe.

In designing the proper profile, therefore, it is necessary to give it such a form that, if a line is drawn through it at any point parallel to the base and the resultant of the forces acting on the mass above such line is determined both with and without water pressure, the maximum intensity of stress in the shorter of the two segments into which the resultant divides the line will not exceed the working stress of the material.

It is possible to design a profile that will not only fulfil the above requirement, but will also be exactly, or very nearly, a profile of "equal resistance"; that is to say, one in which the maximum stresses will be equal at all elevations. The result, however, would be a profile that could not be adopted for an actual structure, because it would conflict with practical constructive features that are of still greater importance than a profile of equal resistance.

Several formulas have been devised for determining a proper practical profile, but such formulas, even when simplified by many preliminary assumptions, are still very complicated and of tedious application. Moreover, as the outcome of all the study that has been given to the subject, a certain type of profile has been evolved to which all designs must very nearly conform; so that at the present day it is needless to go through a series of elaborate calculations, the only result of which must be to reproduce the general type already established. It suffices, therefore, in every particular case, to lay down this general outline and test it at certain elevations, as explained in the following general example.

37. General Illustrative Example.—Let it be required to design the profile of a masonry dam 250 feet high above the surface of the ground, the foundation extending to rock lying 100 feet below the surface. The dam will be built of hydraulic masonry, the average weight of which is 140 pounds per cubic foot. The top width of the dam will be assumed to be 20 feet.

The conditions of the design are that the crushing stress at the base at the level of the ground surface shall not exceed 20,000 pounds per square foot, nor 30,000 pounds 100 feet below, at the bottom of the foundation. It is required, moreover, that at a distance of 100 feet from the top the intensity of stress at the back of the dam, when the reservoir is empty, will not materially exceed 16,000 pounds per square foot, and will increase down to the base, where it may reach 20,000 pounds, as already stated. The pressure at the face, when the reservoir is full, shall be less than the pressure at the back when the reservoir is empty. The batter of either face shall not at any point form an angle of less than 45° with the horizontal. This condition is to prevent the weak edge that would result from a flatter slope. In calculating the water pressure, the surface of the water shall be considered as level with the top of the dam.

This example will be solved by commencing at the top and working downwards, considering first a height of 100 feet, and adding 50 feet successively until the total of 250 feet shall have been reached. As already stated, a length of dam of 1 foot is taken, so that areas in square feet represent volumes in cubic feet.

The first step will be to lay down the right triangle *A B C*, Fig. 18, whose altitude is 100 feet and base $\frac{2}{3} \times 100 = 66.67$ feet. This is surmounted by the small triangle *C D E*, of which the base *C D* is 20 feet, as required. In the actual dam, the face *D E* would have a slight batter, about 1 inch to the foot, but for convenience of calculation it is here drawn vertical.

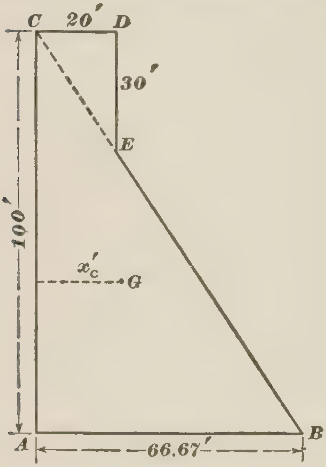

FIG. 18

To find the center of gravity of the figure *A B E D C*, moments will be taken about *C A*. The distance of the center of gravity of the triangle *C D E* from *A C* is one-third the sum of the distances of *D* and *E*, which is $\frac{1}{3}(20 + 20) = \frac{40}{3}$, since these distances are each equal to 20. The area of the triangle *C D E* is $\frac{1}{2} \times 20 \times 30 = 300$ square feet, and, therefore, the moment of this triangle about *C A* is $\frac{40}{3} \times 300 = 4{,}000$.

Likewise, the distance of the center of gravity of *A B C* from *C A* is $\frac{66.67}{3}$, and, as the triangle has an area equal to $\frac{1}{2} \times 66.67 \times 100 = 3{,}333$ square feet, its moment about *C A* is $\frac{66.67}{3} \times 3{,}333 = 74{,}070$. Therefore, the moment of the total area *A B E D C* about *C A* is $4{,}000 + 74{,}070 = 78{,}070$; and, consequently, if the distance of the center of gravity *G* from *C A* is denoted by x_c', we must have

$$x_c' = \frac{78{,}070}{\text{area}} = \frac{78{,}070}{300 + 3{,}333} = 21.5 \text{ feet}$$

This distance gives the shorter segment of the base on which the maximum intensity of stress comes when the reservoir is empty. To obtain the intensity of the stress, the number of square feet in the area, 3,633, must be multiplied by

140 pounds, which gives 508,620 pounds for the weight of the mass above the line *A B*.

The triangle of forces, Fig. 19, is now constructed; it is composed of the vertical line representing the weight of the mass and the horizontal line representing the thrust of the water, which by formula **1** of Art. **26,** is $31.25 \times 100^2 = 312{,}500$ pounds. This thrust intersects the vertical at a distance of $\frac{100}{3}$ feet from the base *A B*, and thus determines the apex of the triangle of forces from which the weight is laid off to scale, if the graphic method is used. The hypotenuse of the triangle of forces determines by its intersection with the base the shorter segment of 24.67 feet on which the maximum intensity of stress comes when the reservoir is full. This intersection can be determined either graphically or by similar triangles; thus,

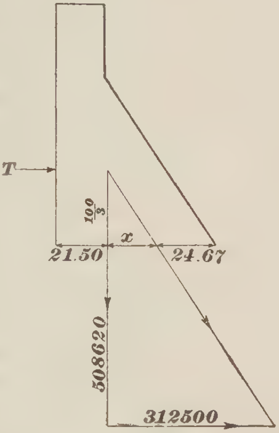

FIG. 19

$$\frac{x}{\frac{100}{3}} = \frac{312{,}500}{508{,}620},$$

whence, $x = 20.5$. Then, the shorter segment is

$$66.67 - (21.5 + 20.5) = 24.67$$

The conformity of this part of the profile with the imposed conditions can now be tested. First, the stress on the segment adjacent to *A*, Fig. 18, when the reservoir is empty, will be considered. In this case, $W = 508{,}620$, $L = 66.67$, $d = 21.50$, and $L - d = 45.17$. Then, by formula **3** of Art. **35,**

$$P = \frac{508{,}620 \times 45.17}{66.67 \times 21.5} = 16{,}028 \text{ pounds}$$

The maximum intensity of stress on the segment adjacent to *B* when the reservoir is full will now be found. Here, $d = 24.67$ and $L - d = 42$, and the remaining quantities *W* and *L* are the same as before; therefore,

$$P = \frac{508{,}620 \times 42}{66.67 \times 24.67} = 12{,}988 \text{ pounds}$$

It is to be noted that it was not necessary to calculate this last stress, because, since the shorter segment was longer than that corresponding to an empty reservoir, it was certain that the stress would be less, which was all that the conditions of the problem required. It is more satisfactory, however, to work out both stresses fully.

38. In discussing the profile so far, two facts become evident: As regards the limiting stress, when the reservoir is empty, the limit has been practically reached; besides, the line of action of the weight, when the reservoir is empty, passes just outside of the middle third of the base *A B*. A study of Fig. 18 will show that this last fact is due to the influence of the small upper triangle. Had the problem been to construct a profile for a dam 100 feet high only, the base *A B* would have been widened a little to the left of *A*, by giving the back of the dam a slight batter of about 1 horizontal to 4 vertical, commencing at a point about 30 feet above *A*. This would give an increased width to the left of *A* of about 7.5 feet, making the total width of base between 74 and 75 feet, instead of 66.67 feet. This would reduce the intensity of stress on the water side considerably, and on the lower side somewhat. Or, if it were important, owing to the great depth of the foundation, or to some other cause, to keep the base as narrow as possible, and, therefore, to keep the back of the dam vertical, an embankment with a berm followed by a flat slope well ripraped would be placed against the back of the dam, rising to a height of about 30 feet, so as to maintain a constant counter-pressure against the back, and thus reduce the stress.

The reason why no change is made when the profile is the upper part of a much higher dam is that, as will be presently seen, the intensities of stress rapidly increase with the height, necessitating a corresponding widening of the base. It is important, therefore, not to begin widening any sooner than is absolutely necessary.

39. A section of 50 feet is now added to the profile. As already mentioned, it is evident that the stresses will increase

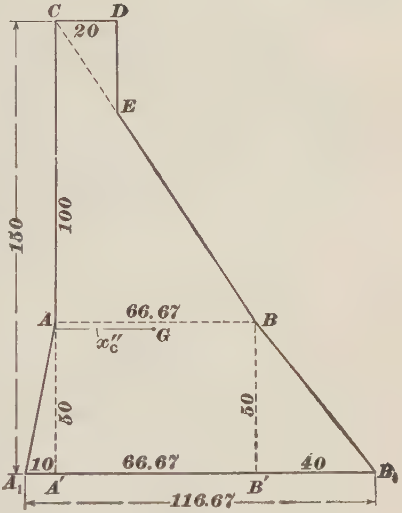

FIG. 20

in a more rapid proportion as sections are added to the height; hence, a batter of 20 per cent. will be given to the back and one of 80 per cent. to the face; that is, a triangle 50 feet vertical and 10 feet base is added to the back, and one 50 feet vertical and 40 feet base to the face, as well as the included rectangle, 50 × 66.67 (see Fig. 20). Throughout this example, $A'C$ will be taken as the axis of moments, distances measured to the right being considered positive, and those measured to the left, negative. The area of the triangle AA_1A' is $\frac{10 \times 50}{2} = 250$ square feet, and the center of gravity is $\frac{10}{3}$, or 3.33, feet to the left of the axis of moments. The area of the rectangle $ABB'A'$ is $50 \times 66.67 = 3{,}333$ square feet, and the center of gravity is $66.67 \div 2 = 33.33$ feet to the right of the axis of moments. The area of the triangle $BB'B_1$ is $\frac{50 \times 40}{2} = 1{,}000$ square feet, and the center of gravity is $\frac{1}{3}(66.67 + 66.67 + 106.67) = 80$ feet to the right of the axis of moments. The static moment of the part ABB_1A_1 about the axis of moments is

$$3{,}333 \times 33.33 + 1{,}000 \times 80 - 250 \times 3.33 = 190{,}256$$

The area of the profile is $3{,}633 + 250 + 3{,}333 + 1{,}000 = 8{,}216$ square feet. The static moment of the part $ACDEB$ is 78,070. The distance x_c'' of the center of gravity of the entire area $A_1ACDEBB_1$ from the axis of moments is, then, given by the equation

$$x_c'' = \frac{78{,}070 + 190{,}256}{8{,}216} = \frac{268{,}326}{8{,}216} = 32.66 \text{ feet}$$

The distance of the center of gravity from the toe A_1 is $32.66 + 10 = 42.66$ feet.

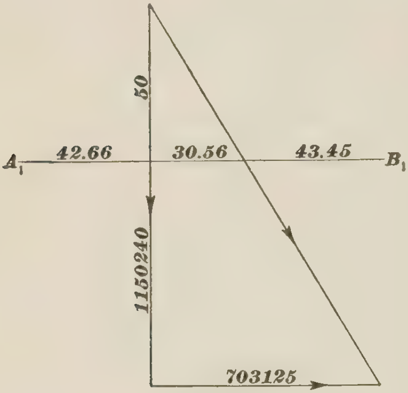

FIG. 21

In Fig. 21, which shows the base $A_1 B_1$ of Fig. 20, the triangle of forces is constructed, the weight of the mass being $8,216 \times 140 = 1,150,240$ pounds, and the thrust of water, $31.25 \times 150^2 = 703,125$ pounds, applied at a height of $\frac{150}{3} = 50$ feet above the base $A_1 B_1$. To ascertain the maximum intensity of stress in the segment adjacent to A_1, when the reservoir is empty, we have $L = 116.67$, $d = 42.66$, $L - d = 74.01$, and $W = 1,150,240$. Substituting in formula **3** of Art. **35**,

$$P = \frac{1,150,240 \times 74.01}{116.67 \times 42.66} = 17,104 \text{ pounds}$$

When water pressure is added, $d = 43.45$, $L - d = 73.22$, and for the maximum intensity of stress on the segment adjacent to B_1, we have

$$P = \frac{1,150,240 \times 73.22}{116.67 \times 43.45} = 16,614 \text{ pounds}$$

This is a reasonable progression in the stresses, and so far shows a satisfactory profile.

40. For the next 50 feet, which will bring the profile up to the height of 200 feet, the base must widen more rapidly. A batter of 30 per cent. will be given to the back, and one of 100 per cent. to the face, as shown in Fig. 22. This adds to the base a triangle of 50 feet altitude and 15 feet base; and to the face, a rectangle of 50×116.67 and a triangle of 50 feet altitude and 50 feet base.

The axis of moments, which is $A'C$ in Fig. 20, is 10 feet to the right of $A_1 A''$, Fig. 22. The area of the triangle A_1A_2A'' is $\frac{15 \times 50}{2} = 375$ square feet, and the center of gravity is $\frac{1}{3}(10 + 10 + 25) = 15$ feet to the left of the axis of moments. The area of the rectangle $A_1B_1A''B''$ is $50 \times 116.67 = 5{,}833.5$ square feet, and the center of gravity is $\frac{116.67}{2} - 10 = 48.33$ feet to the right of the axis of moments. The area of the triangle $B_1B''B_2$ is $\frac{50 \times 50}{2} = 1{,}250$ square feet, and the center of gravity is $106.67 + \frac{50}{3} = 123.34$ feet to the right of the axis of moments.

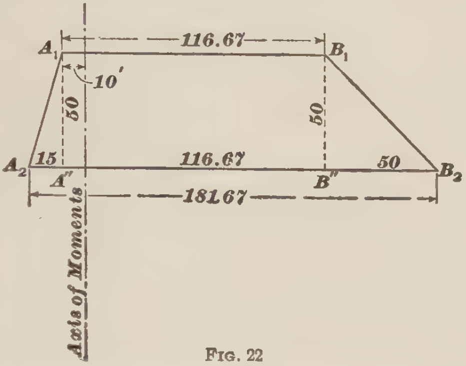

Fig. 22

The static moment, about the axis of moments, of the area represented in Fig. 22, is, then,

$$5{,}833.5 \times 48.33 + 1{,}250 \times 123.34 - 375 \times 15 = 430{,}483$$

The area of the entire profile so far considered can now be found by addition:

$$8{,}216 + 375 + 5{,}833.5 + 1{,}250 = 15{,}674.5 \text{ square feet}$$

For the distance of the center of gravity of the entire profile from the axis of moments we have

$$x_c''' = \frac{78{,}070 + 190{,}256 + 430{,}483}{15{,}674.5} = 44.58 \text{ feet}$$

The distance of the center of gravity from the toe A_2 is $44.58 + 25 = 69.58$ feet.

In Fig. 23, which shows the base A_2B_2 of Fig. 22, the triangle of forces is constructed, the weight of the mass being $15{,}674.5 \times 140 = 2{,}194{,}430$ pounds, and the thrust of the water, $31.25 \times 200^2 = 1{,}250{,}000$ pounds, applied at a height of $\frac{200}{3}$ feet above the base.

To ascertain the maximum intensity of stress in the segment adjacent to A_2, with an empty reservoir, we have

$L = 181.67$, $d = 69.58$, $L - d = 112.09$, and $W = 2{,}194{,}430$. Substituting in formula **3** of Art. **35**,

$$P = \frac{2{,}194{,}430 \times 112.09}{181.67 \times 69.58} = 19{,}460 \text{ pounds}$$

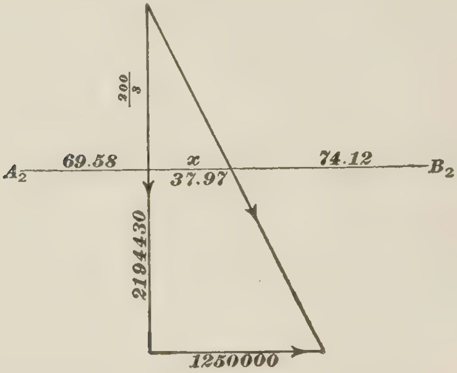

FIG. 23

and, with a full reservoir, on the segment adjacent to B_2,

$$P = \frac{2{,}194{,}430 \times 107.55}{181.67 \times 74.12} = 17{,}527 \text{ pounds}$$

This is also perfectly satisfactory under the given conditions.

41. The next 50 feet will complete the total height of 250 feet. The base is widened at the back, by adopting a batter of 60 per cent. On the face the limiting slope has been reached, so that the only means of widening out is by an offset. Accordingly, a step of 10 feet is made, from which the 45° slope is continued to the base. The additions are shown in Fig. 24.

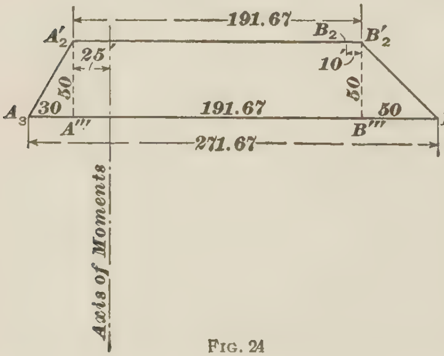

FIG. 24

The area of the triangle $A_3 A_2' A'''$ is $\frac{30 \times 50}{2} = 750$ square feet, and the center of gravity is $\frac{1}{3}(25 + 25 + 55) = 35$ feet to the left of the axis of moments. The area of the rectangle $A_2' B_2' B''' A'''$ is $50 \times 191.67 = 9{,}583.5$ square feet, and the center of gravity is $\frac{191.67}{2} - 25 = 70.83$ feet to the right of the axis of moments. The area of the triangle $B_2' B''' B_3$ is $\frac{50 \times 50}{2} = 1{,}250$ square feet, and the center of gravity is $\frac{1}{3}(166.67 + 166.67 + 216.67) = 183.33$ feet to the right of the axis of moments.

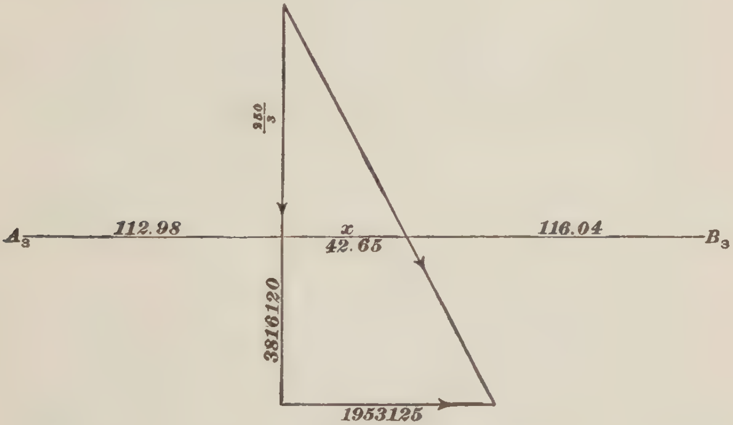

FIG. 25

The static moment of the area represented in Fig. 24 about the axis of moments is

$$9{,}583.5 \times 70.83 + 1{,}250 \times 183.33 - 750 \times 35 = 881{,}711.8$$

The area of the entire profile is now

$$15{,}674.5 + 750 + 9{,}583.5 + 1{,}250 = 27{,}258 \text{ square feet}$$

For the distance of the center of gravity of the entire profile from the axis of moments we have

$$x_c'''' = \frac{78{,}070 + 190{,}256 + 430{,}483 + 881{,}711.8}{27{,}258} = 57.98 \text{ feet}$$

The distance from the toe of the dam is $57.98 + 55 = 112.98$ feet.

All the data necessary for using formula **3** of Art. **35** are shown in Fig. 25. For the stress on the segment adjacent to A_3, $L = 271.67$, $d = 112.98$, $L - d = 158.69$, and $W = 3{,}816{,}120$. Substituting in the formula,

$$P = \frac{3{,}816{,}120 \times 158.69}{271.67 \times 112.98} = 19{,}730 \text{ pounds}$$

For the stress on the segment adjacent to B_3, $L = 271.67$, $d = 116.04$, $L - d = 155.63$, and $W = 3{,}816{,}120$. Substituting in the formula,

$$P = \frac{3{,}816{,}120 \times 155.63}{271.67 \times 116.04} = 18{,}839 \text{ pounds}$$

42. All the conditions of the problem have been complied with, so far as the superstructure is concerned. If, however, it had been desired to go another 50 feet higher, making the total height 300 feet, the base would have assumed extravagant dimensions. It would have been necessary to increase the batter on the back to 80 or 100 per cent. and to take another considerable offset on the face. It is evident, therefore, that, with the given conditions, 250 feet represents nearly the practical limiting height of a masonry dam.

43. Foundation Block.—It now remains to proportion the foundation block. It will be rectangular in section, and the limiting intensity of stress will be 30,000 pounds per square foot. The permissible intensity of stress is raised to this amount because the vertical sides are firmly and squarely compressed by the earth against them, and the masonry can, therefore, safely withstand a greater force. The two cases of a full and an empty reservoir are to be considered. The additional width must be given by front and back offsets.

In determining these offsets in the foundation, it will be best to make them so that the maximum intensity of stress will be the same for a full as for an empty reservoir. This end can be easily attained, since there is now no thrust of the water to be taken into consideration, and the addition consists of a rectangle. Fig. 25 shows that the short segment for an empty reservoir is 112.98 feet from the toe A_3, and 116.04 feet from the toe B_3 for a full reservoir. If an offset of 14 feet is made at the back and one of 11 feet at the front, as shown in Fig. 26, the line of action of the weight of the dam, when the reservoir is empty, will meet the base *MN* of the foundation block at a distance of practically 127 feet from the end *M*, and the line of action of the vertical component of the pressure, when the reservoir is full, will cut *MN* at a distance of 127 feet from the end *N*. The width of the base *MN* is 271.67 + 14 + 11 = 296.67; or, in round numbers, 297 feet. The maximum intensity of stress can now be determined for either a full or an empty reservoir; it is immaterial which condition is assumed, as the intensity of stress is the same for both. Here an empty reservoir will be considered, and moments will be taken about *M*, in order to compute the distance *d* from the end *M* of the line of action of the total weight *W* resting on the base *MN*. The weight of the superstructure, as already determined, is 3,816,120 pounds, and its moment about *M* is 3,816,120 × 126.98. The weight of the foundation block, whose height will be made 100 feet, is 296.67 × 100 × 140, and its moment about *M* is $296.67 \times 100 \times 140 \times \frac{296.67}{2}$. Therefore.

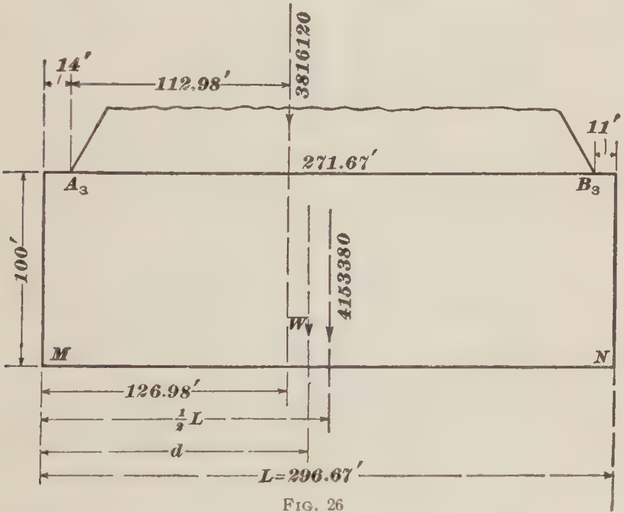

FIG. 26

$$d = \frac{3{,}816{,}120 \times 126.98 + 296.67 \times 100 \times 140 \times (296.67 \div 2)}{3{,}816{,}120 + 296.67 \times 100 \times 140}$$
$$= 138.11 \text{ feet}$$

To apply formula **3** of Art. **35**, the data are: $L = 296.67$,

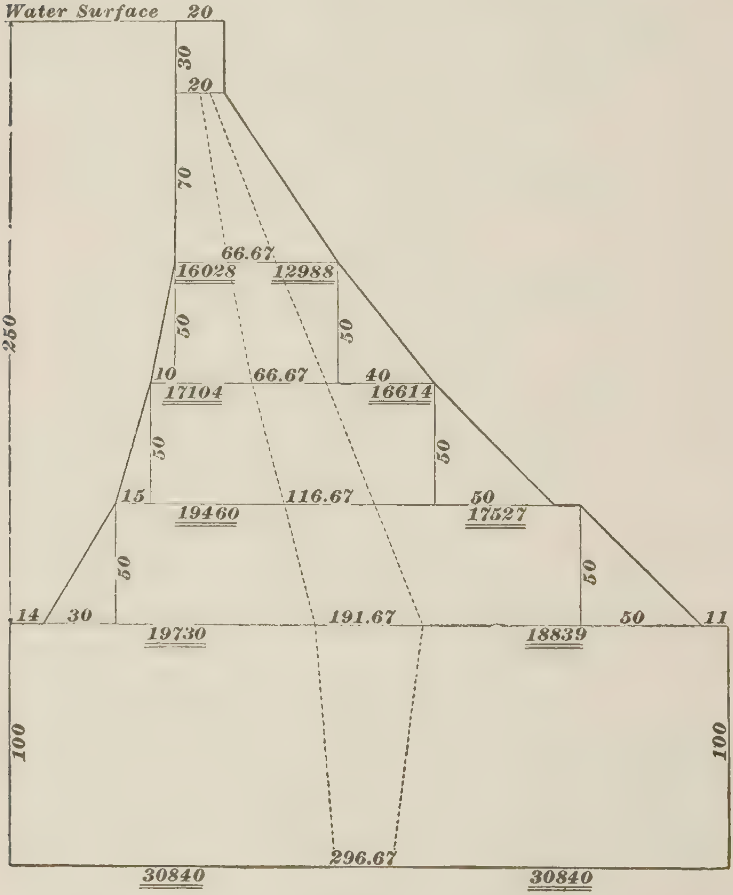

FIG. 27

$d = 138.11$, $L - d = 158.56$, and $W = 7{,}969{,}500$. Substituting in the formula,

$$P = \frac{7{,}969{,}500 \times 158.56}{296.67 \times 138.11} = 30{,}840 \text{ pounds}$$

This slightly exceeds the limit, but in a practical design it would be considered as fulfilling the conditions.

Fig. 27 shows the entire profile with maximum stresses in the corresponding segments, and in dotted lines the "curves of pressure" for an empty and a full reservoir. These curves are constructed by drawing a line through the points where the resultants cut the bases of the various partial profiles, as already determined, except for the upper 30 feet of the profile.

44. General Remarks.—The profile just established is typical, and applies rigorously to a structure weighing 140 pounds per cubic foot, with a top width of 20 feet. Neither the assumed weight per cubic foot nor the top width varies greatly from these values in practice; so that the general form of profile shown in Fig. 27 can be used for any height from 100 to 250 feet, although the dimensions may have to be slightly changed according to the data. This form will always furnish a safe basis from which to start, and each height can be tested with the new data precisely as has been done in the foregoing articles.

45. Accessories of High Dams.—For dams reaching and exceeding 100 feet in height, it is still more indispensable to provide a natural overflow or escape for surplus water than for lower dams. Generally, it is easier to find such an outlet for very high dams than for comparatively low ones, because the water-line approaches so nearly the surrounding summits. Indeed, in the case of high dams, it is frequently necessary to construct one or more auxiliary dams to prevent the water, when it rises to its full height, from escaping over the divides into other valleys. If it should become necessary, however, to allow the waste water to pass over the face of the dam or a portion of it, the profile already established will, in general, suffice for all heights where the base is equal to 90 per cent. of the height. When it falls below this, it should generally be brought up to this percentage by extending the outer toe *B* in the figures. No further general rule will be given, because all such exceptional structures should be considered as special cases, and studied accordingly.

The means of controlling the water is the same in principle in masonry dams of all heights as for earthen dams. They are always simpler, however, because there is no earthen embankment, or at most a smaller one, to penetrate.

46. Execution of Work.—It is not sufficient that hydraulic work should be correctly designed; it is equally important that it should be carefully and properly executed. The closest attention to the most minute details is necessary. To indicate the manner in which the work should be carried on, the following extracts are given from Gould's *Specifications for Dams and Reservoirs*, which embody the specifications followed in the construction of the storage reservoirs of the Scranton Gas and Water Company, of Scranton, Pennsylvania:

(*a*) *Clearing and Grubbing.*—The whole area of the reservoir shall be cleared by cutting all trees, stumps, and bushes even with the ground and removing or burning the same. The area covered by the dam embankment shall be grubbed, so as to remove all stumps, roots, and sods.

(*b*) *Rock Excavation.*— * * * As it is important that, in the rock, the trenches shall be shattered as little as possible, hand drilling and light charges of explosives must alone be used.

(*c*) *Embankment.*—The material for the embankment shall be such as will produce a solid, water-tight bank, and shall be selected subject to the approval of the engineer. It must be taken as far as practicable from the land inside the reservoir. No stones larger than 2 inches in any direction will be allowed in the bank on the upper side of the center wall, or in any slopes exposed to the action of the water, and none larger than 4 inches on the lower side of said center wall. The material shall be laid down in horizontal courses, carried on the work in wagons, or carts, or in barrows, so as to insure its being well traveled over with a view to its consolidation. If, in the judgment of the engineer, these means do not produce a sufficiently compact bank, he may, in addition, order the use of rollers of a design approved by him, or such other means as may be necessary to secure the desired compactness. In all places that cannot otherwise be reached, the bank shall be tamped with heavy rammers. The bank shall be kept thoroughly moistened, while in process of construction, by means of pumps, hose, or sprinkling carts.

(*d*) *Rubble Masonry*.—The masonry for the center wall shall be composed of quarry stones containing not more than one-third of 1 cubic yard each, unless otherwise permitted by the engineer. They shall all have clean quarry faces, beds, and joints, and each stone shall be thoroughly wet just previous to being laid; every stone shall be laid in a full and swimming bed of mortar, and the interior vacancies shall be filled with mortar before any spalls or small stones are put in, the object being to make the wall perfectly water-tight. The bed and end joints of both faces of the wall shall be raked and struck as the work progresses. No stones will be allowed to be deposited nor dressed on the wall, but all stone shall be deposited and dressed on planks furnished by the contractor at his own expense, to prevent the stone from coming in contact with the dirt of the embankment. The center wall shall always be kept at least 2 feet above the adjacent embankment.

The above specifications apply generally to all rubble masonry to be built in this work, with the exception that, for work other than the center wall, larger stones will be permitted, and in some cases required. But all rubble masonry, especially where exposed to the action of the water, shall be strictly hydraulic, as described for the center wall.

The face of all rubble masonry, except the center wall, shall be hammer-dressed, and pitched to true and fair lines, with horizontal and vertical beds and joints, extending back at least 1 foot from the face. No spalls will be allowed in the face, nor any stones less than 3 inches thick. All work will be thoroughly bonded, with a proper proportion of headers, and breaking well the joints.

INTRODUCTION TO CONSTRUCTION DRAWING

GENERAL CONSIDERATIONS

DEFINITIONS

1. Mechanical Drawing.—The general geometrical principles that form the foundation of all instrumental drawing have been given and applied in *Geometrical Drawing*. The application of these principles to the representation of objects is usually called **mechanical drawing.** This term, however, is gradually being restricted so as to cover only the representation of objects connected with mechanical engineering. For this reason, it has been found necessary to apply special terms to the representation of objects in other lines of engineering. For example, the terms *architectural drawing* and *structural drawing* have come into common use to distinguish the branches of mechanical drawing that deal, respectively, with building and structural work. Architectural drawing consists of the representation of the general appearance, form, and design of a structure; few dimensions other than principal dimensions are given, the accuracy of the drawing being relied on to show the required sizes of parts. Structural drawing, on the other hand, deals with the structural features; all necessary dimensions are given, including the required sizes and arrangement of all parts. In addition, structural drawing usually includes the representation of all details.

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2. Construction Drawing.—All drawings relating to civil-engineering construction come under mechanical drawing, taking the latter term in its broader sense. For convenience, however, drawing, when applied to civil-engineering work, will here be called **construction drawing.**

DRAWINGS

3. Classification of Drawings.—Drawings are usually classified, according to the purpose for which they are to be used, into *design drawings*, *detail drawings*, and *shop*, or *working*, *drawings*.

4. A **design drawing,** as the name implies, represents the general design, and gives the general arrangement and dimensions, of the finished work. This kind of drawing requires less work in preparation than any other, and is, therefore, not nearly so expensive. It is much used to show proposed work, sometimes as many as four or five designs being shown.

5. When the general design is adopted, more complete and elaborate drawings, called **detail drawings,** are made. They show all the information that is given on the adopted design drawing, and, in addition, the arrangement of all details. Sufficient dimensions and information are given on these drawings to compute all dimensions that are not directly given.

6. Detail drawings are often used by workmen in construction, but better results are obtained, and less trouble is encountered, if **shop, or working, drawings** are made. These drawings contain all the information usually given on design and detail drawings, and, in addition, all the actual dimensions of each separate piece that enters into the construction of the work. It is customary, in making working drawings, to make a general drawing showing the location of each part in the finished work, and also a separate drawing of each part, giving all the information that the workmen need for its construction. In making a working drawing, the draftsman should keep in mind the fact that the drawing is to be used by workmen, whose knowledge of drawing is usually very limited; he should, therefore, endeavor to make it so plain that there will be no possibility of confusion or misunderstanding. The draftsman should consider the working drawings as a complete letter of instruction to the workman, telling him what material to use, what sizes to make the parts, and how to finish them. Working drawings should be so complete that it will not be necessary for workmen to lose time going to the drafting room for further information.

7. Paper Drawings.—Construction drawings are first made on paper in pencil, and the lines are not inked. For this purpose, a good grade of Manila paper on which lines can be easily drawn and erased should be used. A 4-h (HHHH) or 6-h (HHHHHH) pencil is well adapted to this kind of work. The lines should be made somewhat heavier than on drawings in which the lines are to be inked. These drawings are called **original,** or **pencil, drawings,** and are usually made complete, all information being given. They are sometimes filed away as records, but are usually destroyed after tracings have been made.

8. Tracings.—When the pencil drawings are finished, they are reproduced by tracing the lines on specially prepared cloth, called **tracing cloth.** Tracing cloth is linen treated with a waxy sizing so as to make it semitransparent. After it is sized, one side is dull and slightly rough, and the other is bright and smooth, presenting a glazed surface.

In copying a drawing on tracing cloth, the cloth is tacked over the drawing, and the lines on the cloth are drawn by following those on the drawing, which are plainly seen through the cloth. A drawing thus copied on tracing cloth is called a **tracing.** Tracings can also be made on **tracing paper,** which is transparent paper especially prepared for this purpose. When, however, a tracing is to have much handling, cloth is preferable to paper.

Some tracers prefer to draw on the dull, rough side, and others prefer the smooth side of tracing cloth. Many offices adopt one side for all their draftsmen to use. The smooth side has the advantage that erasures are more easily made; however, it does not take ink so well as the rough side. When the smooth side is used, powdered chalk should be spread and rubbed over it; this removes the gloss from the surface of the cloth, which then takes in ink more easily. The student is advised to practice the use of both sides. For this purpose, he may draw in pencil some simple figures, such as triangles, rectangles, and circles, and make tracings, using sometimes one side of the cloth, sometimes the other.

9. Stretching Tracing Cloth.—In construction drawing, all shading is omitted, the surface or boundary lines being about the same thickness at every part of the drawing. Since tracing cloth is only semitransparent, the pencil lines on the paper drawing should be comparatively heavy, so that they will show through the tracing cloth when the latter is placed over them. In placing a sheet of tracing cloth over a pencil drawing, care should be taken that the pencil drawing is stretched tight and smooth on the drawing board, and that the tracing cloth is so stretched and tacked down on top of the paper drawing that there is no unevenness. In case there are any wrinkles in the cloth after it is laid, they can be removed by taking out one or two tacks at a time and smoothing the cloth outwards with the palm of the hand, replacing each tack as a part of the cloth is smoothed.

BLUEPRINTING

10. Blueprints.—When tracings are finished, as many copies of them as are wanted are obtained by a special process called **blueprinting.** These copies are called **blueprints,** and are taken on **sensitized paper.** This paper is coated on one side with a chemical preparation that can be removed at any time, if it has not been exposed to bright daylight or sunlight, by immersing the paper in water; after a short washing, the coating is entirely removed and the paper resumes its original color. If the coating is exposed to bright daylight or sunlight, it undergoes a chemical change that colors the surface of the paper, and it is then impossible to remove the color by washing. This principle is made use of in blueprinting. The tracing is placed on top of a sheet of sensitized paper in such a way that the coated surface of the paper, and the ink lines on the tracing, are uppermost. A sheet of glass is then placed on top of the tracing, and the whole is exposed to the brightest available light. That part of the coating on the sensitized paper that is directly under the ink lines on the tracing is protected from the light, and undergoes no change; all other parts undergo the change referred to, the color of the surface of the paper being changed. When the drawing has been exposed for a sufficient length of time, it is removed from the light; the sensitized paper is at once immersed in a large basin or sink containing clear water, and thoroughly washed. The coating is washed off all that part of the paper that was covered by the ink lines, leaving the paper white; all the remainder of the paper is colored. The result is an exact reproduction of the tracing. As many prints as desired can be made, one at a time, from a tracing.

11. Sensitized Paper.—Sensitized paper can be purchased in rolls of almost any desired width and length all ready for use. There are several kinds on the market, which differ only in the color of the finished print and in the method of washing they require: the one most used and most economical is *blueprint paper*, so called because all parts of the print, except the lines, notes, and dimensions are blue. Blueprint paper should be kept in a closed tin tube or in a drawer in a darkened room. It should under no circumstances be handled in the full light of day until after it has been washed.

12. Directions for Making Blueprint Paper.—It is sometimes desired to prepare instead of buying the sensitized paper. This is usually done as follows: Dissolve 2 ounces of citrate of iron and ammonia in 8 ounces of water; also, 1¼ ounces of red prussiate of potash in 8 ounces of water. Keep the solutions separate and in dark-colored bottles in a dark place. Better results will be obtained if $\frac{1}{2}$ ounce of gum arabic is added to each solution. When ready to prepare the paper, mix equal portions of the two solutions, and be particularly careful not to allow any more light to strike the mixture than is absolutely needed. For this reason, it is necessary to have a dark room in which to work. There must be in this room a tray or sink of some kind that will hold water; it should be larger than the blueprint and about 6 inches deep. There should also be a flat board large enough to cover the tray or sink. There must be an arrangement like a towel rack to hang the prints on while they are drying; this arrangement will be called a **print rack.** The paper used for blueprinting should be a good, smooth, white paper, and may be purchased of any dealer in drawing materials. Cut it into sheets a little larger than the tracing, so as to leave an edge around it when the tracing is placed on it. Place eight or ten of these sheets on the flat board before mentioned, taking care to spread them flatly one above another, so that the edges do not overlap. Secure the sheets to the board by driving a brad or small wire nail through the two upper corners sufficiently far into the board to hold the weight of the papers when the board is placed in a vertical position. Lay the board on the edges of the sink, so that one edge is against the wall and the board is inclined at an angle of about 60° with the horizontal. Darken the room as much as possible, and obtain whatever light may be necessary from a lamp or gas jet, which should be turned down very low. With a wide camel's-hair brush or a fine sponge, spread the solution just prepared over the top sheet of paper. Be sure to cover every spot, and do not get too much on the paper. Distribute the solution as evenly as possible over the paper, in much the same manner that the finishing coat of varnish would be put on by a painter. Remove the sheet by pulling on the lower edge, tearing it from the nail that holds it, and place it in a drawer where it can lie flat and be kept from the light. Treat the next sheet and each succeeding sheet in exactly the same manner, until the required number of sheets have been prepared.

Unless a large number of prints are constantly used, it is cheaper to buy the paper already prepared. There is scarcely anything saved in preparing the paper, and better results are usually obtained from the commercial sensitized paper, since the manufacturers have machines for applying the solution, and are able to distribute it very evenly.

13. **Directions for Making Blueprints.**—In Figs. 1 and 2 are shown two views of a printing frame that is well adapted to sheets not over 17 in. × 21 in. The frame is placed face downwards, and the back *A* is removed by unhooking the brass spring clips *B*, *B*, and lifting it out. The tracing is laid on the glass *C*, with the inked side touching the glass. A sheet of the prepared paper, perfectly dry, is laid on the tracing with the sensitized side downwards. The paper and tracing are smoothed out so that they will lie perfectly flat on the glass, the cover *A* is replaced, and the brass spring clips *B*, *B* are sprung under the plates *D*, so that the back cannot fall out. While all this is being done,

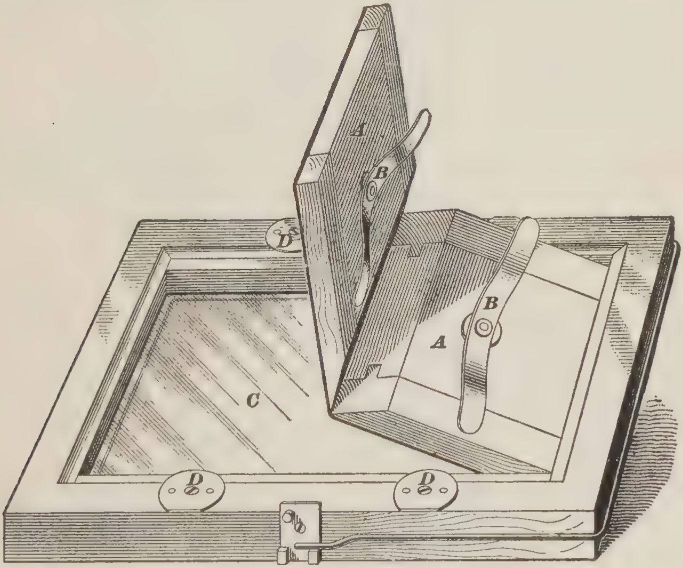


FIG. 1

the paper should be kept from the light as much as possible. The frame is now placed where the sun can shine on it, and is adjusted, as shown in Fig. 2, so that it will get the most light, or, if the sun is shining, so that the sun's rays will fall on it as nearly at right angles as possible. According to the conditions of the sky—whether clear or cloudy—and the time of the year, the print must be exposed from 3 to 15 minutes. The tray, or sink, already mentioned should be filled to a depth of about 2 inches with clear water (rain water if possible). The print having been exposed the proper length of time, the frame is carried into a dark part of the room, or the curtains at the windows are drawn, and the print is taken out of the frame. The print is then placed in the water with the sensitized side down, care being taken that the paper lies flat and that there are no air bubbles under it. The print is allowed to soak for from 3 to 6 minutes, ordinarily while the next print is being put in the frame. The print is then turned over in the water and laid down flat with the blue side up. Water is then splashed over it with the hands so as

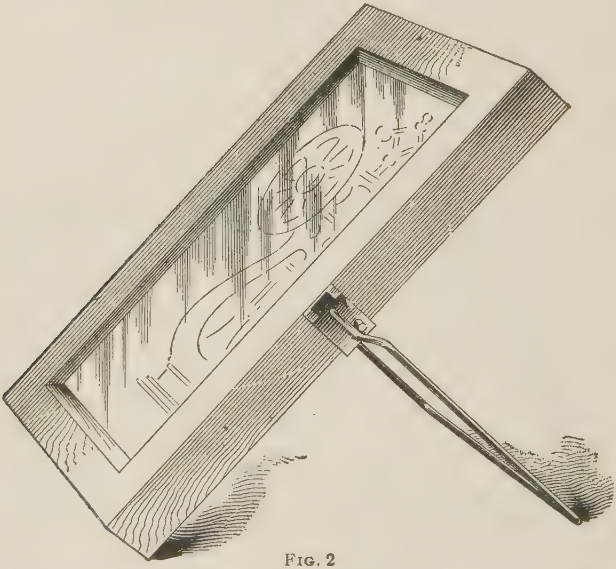


FIG. 2

to thoroughly rinse the chemicals from the surface. The print is then taken from the sink and suspended by its upper edge to dry on the print rack. The proper duration of the bath can best be determined by observation; dark purple- or bronze-colored spots on the prints indicate that the prints were not washed thoroughly on those spots. The prints should be again immersed in the water and washed. There is very little danger that the bath will be too long.

14. Time of Exposure.—The most important point in connection with blueprinting is the time of exposure. If a print is exposed for too short a time, the background or blue portion is very pale, and it is difficult to distinguish the lines and letters. If the print is exposed for too long a time, the background becomes dark blue or purple, and the light penetrates through the ink lines, thereby coloring the paper where it should remain unaffected. The time of exposure varies with the weather conditions, being less on a bright sunshiny day than on a dark cloudy day. On a cloudy day, when the sun is alternately visible and obscured, it may take several times as long for one print as for another. It is best to judge the proper time of exposure to the light by the color of the strip of print projecting beyond the edge of the tracing. To obtain the exact shade of the projecting edge, take a strip of blueprint paper about 12 or 15 inches long and 3 or 4 inches wide. Divide it into twelve or fifteen equal parts and mark them on the back 1, 2, 3, etc. Take a piece of tracing cloth, and place the blueprint paper and cloth in the frame in such a way that the cloth covers one-half the width of the paper. Now expose the paper to the light, and at the end of 1 minute cover the part of the strip marked 1 with a thin board or anything that will prevent the light from striking the part covered. At the end of the second minute, cover parts 2 and 1; at the end of the third minute, parts 3, 2, and 1; etc. Part 1 will have been exposed 1 minute; part 2, 2 minutes; part 3, 3 minutes; etc. When the whole strip is covered, remove the blueprint paper, cut the part that was under the tracing cloth from the remainder, and wash the former thoroughly. When it has dried, select a good rich shade of blue and notice the number of this section on the back. Now observe carefully the color of the same portion of the remainder of the strip; this is the desired color of the protruding edge and should be kept in mind. All prints should be exposed until the protruding edge attains this color.

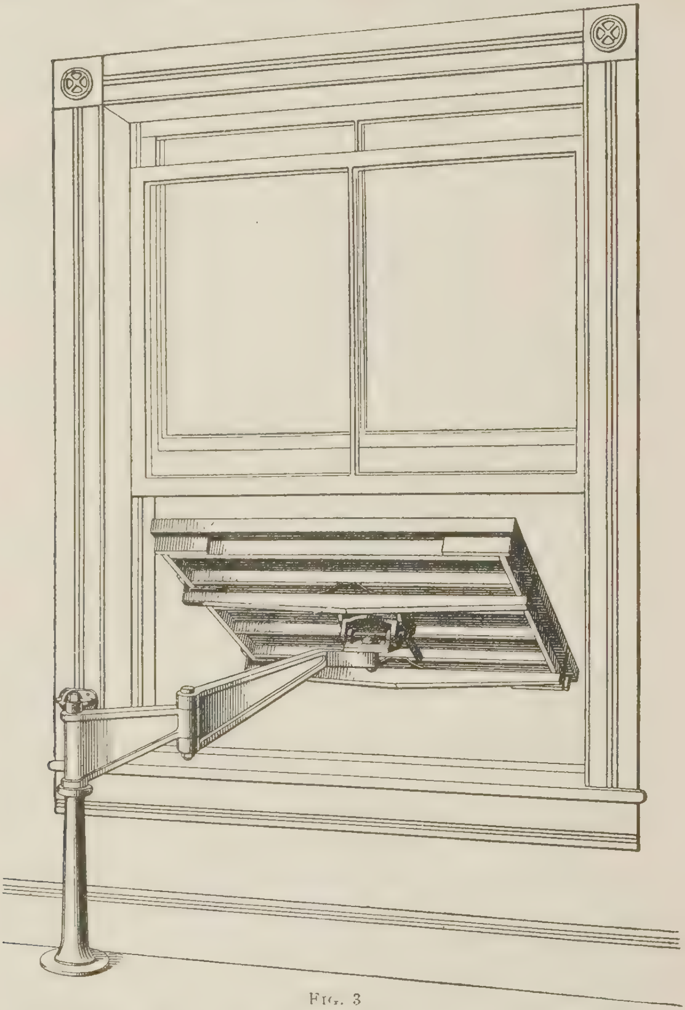


FIG. 3

15. Large Frames.—As stated in Art. **13,** the type of frame shown in Fig. 1 is well adapted to small-sized drawings. For large drawings, the same general type of frame is employed, but it is usually supported by some device. In Fig. 3 is shown a patented frame that can be placed outside the window and adjusted to any angle. When not in use, it can be folded up against the wall. It is made in different sizes, from 16 in. × 24 in. to 48 in. × 72 in.

16. Rolling Frames.—The larger sizes of blueprint frames are too heavy for convenient handling, and it is customary to mount them on a support, as shown in Fig. 4. The support is mounted on rollers *a*, *a*, and runs on a track *b* that extends through a window opening into the outer air. The frame *c* is shown bottom up in the figure, having been run out this way. It can be turned to any angle by hand and held at that angle by means of the pawl and ratchet *d*. This form of frame and support is very serviceable.

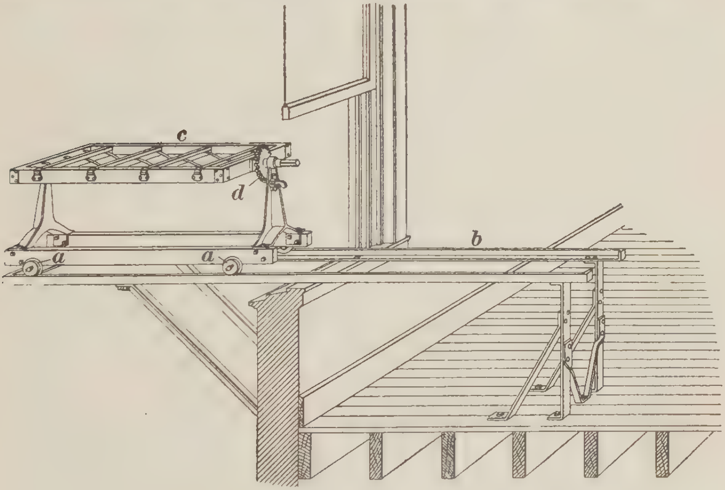


FIG. 4

17. Electric-Light Process.—Recently, an apparatus has been perfected for taking blueprints by electric light. The device generally consists of a glass cylinder properly mounted and provided with a canvas cover, or sheet, that is wrapped, or drawn, around the outside, and in this way holds the tracing and sensitized paper against the glass. When set in this manner, an arc light is slowly lowered inside the cylinder. When the light reaches the bottom of the cylinder, the print is taken out and washed. The time of exposure of the print is regulated by the speed of the descending arc, which is controlled by some mechanical device.

18. Blueprint Cloth.—Blueprints are frequently made on cloth instead of on paper. The process is the same as for printing on paper. Cloth blueprints are more expensive than paper blueprints, but for working drawings that will be much handled by the workmen, they are better, as they stand wear and tear much better than paper prints.

DRAFTSMEN

19. Grades of Draftsmen.—In a well-equipped engineering office, where a large number of draftsmen are employed, it is customary to divide them, according to their work, into several classes. The usual classes are *designers*, *detailers*, *tracers*, and *checkers*.

20. Designers are the most proficient, and although many of them do very little drafting, they are classed under the head of draftsmen. They design the work and compute the strength and required sizes of all parts. In some drafting rooms, particularly when an unusual piece of work is under way, the designer makes numerous rough sketches illustrating the general arrangement of parts, and gives them to the detailer to aid him in the preparation of the drawings. In addition to having a knowledge of mechanical drafting, the designer must be familiar with the engineering principles involved in the construction of the work he designs.

21. Detailers prepare the design, detail, or working drawings, as the case may be, from information and instructions given by the designers. They compute all secondary dimensions, such as rivet spacing, thickness of retaining walls and dams at different heights, etc. from the principal dimensions furnished by the designer. Detailers should have a thorough knowledge of mechanical drafting, and should be well acquainted with the standard details of the types of construction in which they are engaged.

22. Head Draftsman.—In almost all drafting rooms there is a head, or chief, draftsman, who supervises the work of the detailers and tracers. He receives general information and instructions from the designers, which he transmits to the detailers, adding small details and special instructions to expedite the making of the drawings. He gives to each detailer, as far as possible, the class of work to which the detailer is best adapted, either from natural aptitude or from previous experience.

23. Tracers.—Detailers usually make their drawings in pencil on paper. The drawings are then turned over to less experienced men to be traced. These men are called **tracers.** Their knowledge need not extend beyond familiarity with the use of instruments. Tracers should be neat, accurate, and rapid workers, and fairly good letterers. A draftsman in this position has very good opportunities to become acquainted with the best types of construction, and, if he has an aptitude for drawing and some knowledge of engineering principles, can advance very rapidly

24. Checkers.—When tracings are completed, they are turned over to a man to be carefully examined for mistakes. This man is called a **checker,** and usually examines the work of the designer, detailer, and tracer. When the drawings are checked, the checker places his initials on them, and thereafter he is responsible for their correctness. The checker is usually a man that has had experience as a tracer, detailer, and designer, and has demonstrated that he can be relied on for accuracy and faithfulness. In spite of the utmost care, designers, draftsmen, and tracers make mistakes, and it is the duty of the checker to find them. He should never assume or take for granted that anything on the drawings is correct. The checker marks on the tracings whatever mistakes he finds, and then hands the tracings to the tracers for correction.

25. Small Drafting Forces.—In many offices where few draftsmen are employed, the positions outlined in the preceding articles are frequently combined. For example, designers are sometimes also detailers, and may trace their own drawings. More seldom, draftsmen may be found that design, draw, trace, and check their own work. This course is very objectionable: to avoid mistakes, at least two men should look over each drawing.

26. Blueprint Clerk.—A position that does not strictly come under the head of draftsman, but is closely allied to it, is that of **blueprint clerk**, or **blueprint boy**. In all large engineering offices, blueprints are made by one man, in order to avoid confusion and loss of time. It is customary to turn the tracings over to the blueprint clerk with a note as to the number of prints required. He returns the tracing with the required number of prints when they are dry. Almost any one can become proficient in blueprinting in less than 1 day, no special talent being required—nothing in fact but a little common sense. The position is mentioned here because the blueprint clerk has a good opportunity to become acquainted with different types of construction. If he is attentive and ambitious, and has some knowledge of drawing, he will soon know how to read plans and may secure a position as tracer.

SCALES

27. Scale of a Drawing.—It is seldom convenient to make full size the drawing of an object. The ratio of the dimensions on a drawing to the corresponding dimensions of the object represented is called the **scale** of the drawing. Thus, if every line in the drawing is one-fourth of the corresponding line on the object, the scale of the drawing is $\frac{1}{4}$, or 1 to 4. Since, in such a case, every foot on the object is represented by 3 inches on the drawing, this scale is also expressed as **3 inches to the foot.** Similarly, a scale of $1\frac{1}{2}$ inches to the foot is one in which a distance of 1 foot on the object is represented on the drawing by $1\frac{1}{2}$ inches. This scale is also expressed as a $\frac{1}{8}$ scale, since the ratio of any line on the drawing to the corresponding line on the object is $\frac{1}{8}$.

In some cases, the drawing is made to a scale greater than 1; that is, the lines on the drawing are larger than the corresponding lines on the object. This is usually the case in the representation of very small objects or details.

The dimensions given on a drawing are always followed; but it is customary to state the scale to which drawings are made, as this adds to the clearness of the drawing. It is common practice to have parts on the same sheet drawn to different scales, in order to make every part sufficiently clear.

28. Measuring Scales.—The name *scale* is also given to a graduated rule used for measuring distances. Some of these rules are graduated to a **natural scale,** that is, into true inches and fractions of an inch, such as eighths, sixteenths, tenths, etc. Others are graduated in such a manner as to give directly, without computation, the proper reduced lengths of lines drawn to the scale adopted for the drawing.

Fig. 5 shows a convenient scale, which combines eleven systems of subdivision, and may be used for all the mechanical drawing ordinarily done in a drafting room. This scale is triangular in section, and its graduations cover a length of 12 inches. On each of its edges is laid off a scale, as shown at *A*, *B*, and *G*. The scale at *G* is **full size,** or natural; this edge of the scale is divided into inches and fractions of an inch down to sixteenths, and is used for drawings in which an object is represented in its natural size. On the opposite side, at *B*, is shown the quarter-size scale of 3 inches to the foot. This is indicated by the *3* at the end. The first 3-inch (actual size) division from *B* to *C* is subdivided into twelve parts representing inches, and each of these parts is then subdivided into equal parts, generally to represent fractions of an inch. From *C* to *D*, *D* to *E*, and *E* to *F*, the scale is marked in its

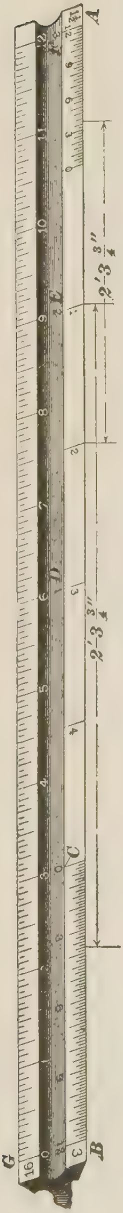


FIG. 5

main divisions to represent 1 foot each, each foot being represented on the scale by an actual length of 3 inches. From *A* to *B*, the scale is independently divided into spaces of $1\frac{1}{2}$ inches (actual size) to form an eighth-size scale, or $1\frac{1}{2}$ inches to the foot; this is indicated by the $1\frac{1}{2}$ at the end. The divisions of the latter scale occur on and between the marks for the 3-inches-to-the-foot scale.

The other sides and edges of the instrument are divided into scales of 1 and $\frac{1}{2}$, $\frac{3}{4}$ and $\frac{3}{8}$, $\frac{1}{4}$ and $\frac{1}{8}$, and $\frac{3}{16}$ and $\frac{3}{32}$ inch to the foot, each scale being designated by the fraction at its end. Different makers do not always arrange their scales in the same manner. Thus, instead of having a full-size scale and scales of 3 inches to the foot and $1\frac{1}{2}$ inches to the foot on one side, as shown in Fig. 5, some makers have the full-size scale, and the scale of $\frac{3}{16}$ and $\frac{3}{32}$ inch to the foot on one side. It will be noticed that the numbering of the feet on these scales does not start at the end of the instrument, but at the first main division from the end. Thus, on the quarter-size scale, or scale of 3 inches to the foot, the zero mark is placed at *C*, and the first foot is measured to *D*. This is done so that the feet and inches may be laid off independently and with one reading of the scale.

The figures indicating the number of feet on this scale are placed along the inside edge at *D*, *E*, and *F*, the numbers running in a direction away from the part containing the inches. The numbers indicating inches run in an opposite direction from those defining the feet.

29. Laying Off Distances to Scale.—To lay off 2 feet $3\frac{3}{4}$ inches on a scale of 3 inches to the foot, and from a given point, place the scale on the point so that the 2-foot mark will be directly over it; then from the zero mark *C* lay off 3¾ inches as shown, locating a second point. The length of the distance thus laid off between the two points represents 2 feet 3¾ inches to the scale of 3 inches to the foot. The scale of 1½ inches to the foot is used in a similar manner to lay off the same distance. The figures indicating feet on this scale are placed nearer the edge, in order to prevent confusion in reading.

To draw to half size, or to a scale of 6 inches to the foot, use the full-size scale, and remember that every ½ inch on that scale corresponds to 1 inch on the object—that is, that every dimension on the drawing is only one-half of the real length. Thus, to lay off 5⅞ inches, lay off 5 half inches and 7/16 inch over; the result represents a line 5⅞ inches long to a scale of 6 inches to the foot.

30. Special Scales.—It may happen that a draftsman is obliged to make a scale, when the size of his plate is limited and a general drawing of some object is desired. In such a case, one scale may be too large for the drawing to be made on a sheet of the required size; another scale may make the drawing too small. For example, a ⅛ scale may be too large and a 1/16 scale too small; a 1/10 scale may be just right. If the draftsman has no 1/10 scale, he may make one by taking a piece of heavy drawing paper and cutting out a strip about the size of an ordinary scale and laying off 1 foot on it, dividing it into ten equal parts. Each division or part will represent 1 foot on the object. Divide one of the end parts into twelve equal parts, and each will represent 1 inch on the object. Lines indicating half and quarter inches may be drawn if considered necessary.

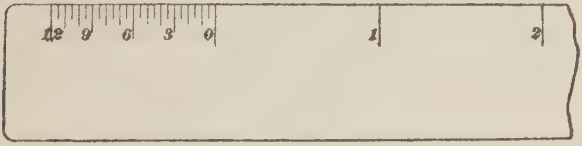


FIG. 6

Fig. 6 shows part of a scale made in this manner, giving feet, inches, and half inches—the quarters, eighths, etc. of an inch being judged by the eye.

To make a $\frac{1}{5}$ scale, lay off 12 inches and divide this distance into five equal parts. Divide one of the end divisions into twelve equal parts, to represent inches, and then divide each of these parts into halves, quarters, eighths, etc., as far as desired.

DIRECTIONS FOR DRAWING THE PLATES

GENERAL INSTRUCTIONS

NOTE.—The general instructions that follow apply to all the Drawing Plates given in the present and in succeeding Sections.

31. Size of Plates.—The plates are to be of the same size as those drawn in connection with *Geometrical Drawing;* that is, 14 in. × 18 in., with border lines drawn $\frac{1}{2}$ inch from the edge, making the working space 13 in. × 17 in.

32. Title and Number of Plate.—The title or name of each plate is, when convenient, to be placed at the lower right corner of the sheet; a space of $1\frac{1}{2}$ in. × 4 in. is to be reserved for this purpose. In addition to the title, there will be given in this space the number of the drawing, the draftsman's name, and the date when the drawing was finished. If desired, the date when the drawing was started may also be given.

33. Abbreviations.—In general, abbreviations are used on drawings only when lack of space prevents the use of complete words. There are, however, a few abbreviations that can be used without hesitation, having been fixed by long practice, such as D. or "Diam." for diameter; R. or "Rad." for radius; "Thds." for threads; f. for finish; C. L. or ℄ for center line.

34. Dimensions.—As a rule, all dimensions above 12 inches are to be given in feet and inches; the feet being designated by a single accent mark (′), and the inches by a double accent mark (″). The feet and inches should be separated by a short dash; thus, 2′–3″.

Fractions should be written with the dividing line between the numerator and the denominator in a horizontal position, thus: $\frac{3}{4}''$, $\frac{7}{16}''$; never with the dividing line vertical or inclined, thus 3|4″, 7/16″. Mixed numbers should be written without a dash between the whole number and the fraction; thus: $4\frac{1}{2}''$, $5\frac{3}{4}''$. In writing decimals less than 1, a zero should generally be placed before the decimal point to avoid confusion; thus, 0.15″, 0.25″, not .15″, .25″.

Where possible, dimensions should be written so that they can be read from the bottom and the right side of the drawing.

35. Notes on Drawings.—When it is desired to convey special information, this is done by writing notes on the drawings. The location of notes should be chosen with great care, so that they will not be obscured by the lines of the drawing. The notes should be as clear and concise as it is possible to make them without omitting necessary information.

36. Drawing the Plates.—The method of making the drawings in this and succeeding Sections is different from that followed in *Geometrical Drawing*. The plates will first be made in pencil on Manila paper, but will not be inked in. The pencil lines should be about 50 per cent. heavier than those in the *Geometrical Drawing* plates. This is to render them more clearly visible through the tracing cloth (see Art. **9**). All lettering should be done on the pencil drawing before tracing, so that it will be properly and conveniently arranged.

37. Making the Tracings.—When the student finishes a plate in pencil, he should compare it carefully with the one sent by the Schools, and read again the directions given in the text. When he is certain that his plate has been drawn correctly, he should spread the tracing cloth over it, and stretch it smooth and flat, as described in Art. **9.** If the smooth side of the cloth is used, it should be thoroughly rubbed with powdered chalk, magnesia, or pumice stone (any of which can be obtained at a drug store for a few cents), in order to remove all grease and dirt, and to slightly lessen the gloss on the surface, so the ink will flow evenly. When the entire surface has been rubbed, the remaining powder can be dusted off with a clean handkerchief or other cloth. Be careful not to use a dirty cloth, especially one that is likely to contain grease. Next fill the right-line pen and adjust it to the proper width of the line, trying it on the edge of the tracing cloth outside of the boundary lines, until it is right and the ink flows smoothly. Now, *without delay*, start drawing the lines on the tracing. Any delay allows the ink to dry and harden at the point of the pen, and this makes it necessary to clean out the pen and refill it. It is best to begin with the horizontal lines, then ink the vertical lines, and finally the inclined lines. When there are curves tangent to straight lines, the curves should be traced first. In this way, a better junction can be made than if the straight lines are drawn first. It should be remembered that ink dries slowly on tracing cloth, so that care must be taken not to smudge lines by rubbing them before they are dry. *Lines must be allowed to dry by themselves; they should not be dried with blotting paper*, as this paper absorbs part of the ink and leaves a pale line that does not show well on the blueprint. After a little experience, the draftsman will learn to divide his work in such a manner that he can work continually, moving from one part of the drawing to another when the lines are dry. Lettering, dimensions, and titles should not be inked in until all the lines are drawn.

38. **Method of Handling the Ruling Pen.**—In drawing a line, the pen should not press on the cloth very hard, and should move with almost uniform speed, as this gives a smoother line. Care should be taken to start a line at the right point, and not to carry it beyond nor stop it short of the end. At first, it may be found somewhat difficult to get lines that are smooth, distinct, and uniformly black; but,

after a little practice, this difficulty will disappear. It should be kept in mind that, in order to obtain a good line, it is not necessary to press the pen hard on the cloth. Ink work is entirely different from pencil work in this respect. It is sufficient to press the pen lightly, so that it will be well in contact with the surface of the cloth, to make the ink flow smoothly. An even pressure is necessary, however; good lines cannot be obtained if the pen just touches at one spot and digs in another. Never have the nibs of the pen in contact, even for the finest lines, as smooth lines cannot then be drawn. After a little practice, the tracer learns to get a fine line by spacing the nibs a very small distance apart. Each time the pen is filled, the width of line should be tried on the edge of the sheet of tracing cloth to see if the nibs need to be readjusted.

When it is necessary to draw pencil lines on a tracing, they should be made with a soft pencil, preferably softer than HH, as hard pencils cut into the cloth and the lines are difficult to erase.

39. Erasing.—In case it is necessary to erase an ink line from a tracing, this can be done by means of an ink eraser. Do not try to remove the line quickly by rubbing hard, but rub gently until the line is removed. Then rub some chalk on the surface where the erasure has been made. If the ink runs through the cloth, wait until the tracing is finished, then turn it over and remove the ink by means of the ink eraser. Have a blotter constantly at hand, so that if a blot occurs it can be removed at once. Do not lay the blotter flat on the paper on top of a blot, but first dip the corner of the blotter into the ink and allow it to absorb all the ink it will. Then place the blotter on top of the spot, and press it down gently. What remains of the blot after this can be entirely removed by means of the ink eraser, care being taken to allow the blot to dry entirely before the eraser is used.

40. Erasers.—The kind of rubber to be used varies somewhat with the nature of the material on which the drawing is made If bristol board, tracing cloth, or paper with a coarse grain or fiber, which is apt to raise by rubbing, is used, a soft rubber is recommended. On duplex, double elephant, and the ordinary drawing papers, a harder rubber will be found better.

For erasing ink lines, ink erasers are most satisfactory. They are made with emery or glass mixed with the rubber; the action of the eraser cuts the outer skin of the paper on which the lines are drawn and removes the ink. If there should be two or three erasures over the same spot, the paper or cloth is very apt to rub through. With double elephant or any good drawing paper, this is not so likely to occur with two or three erasures; but when a line is to be erased from tracing cloth care must be exercised, because the cloth will not stand much rubbing without wearing through.

Before attempting to remove a line, the eraser should be cleaned by rubbing it on a clean piece of paper. Should it then smear the ink, it is probable that it is too hard or old, and it should be discarded. A rubber of the size and shape shown in Fig. 7 is usually found to be most convenient.

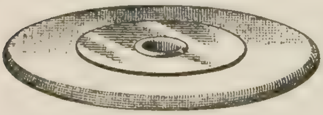


FIG. 7

When erasing on tracing cloth, the cloth should be laid on a smooth surface and held firmly, the rubbing being done gently and patiently until all traces of the lines have disappeared. A rotary motion imparted to the eraser often facilitates the operation. The gloss of the tracing cloth, which may have been destroyed by rubbing, can be renewed by applying powdered pumice stone with a chamois skin and finishing the spot with chamois alone. In erasing ink lines from drawing paper, the gloss may be brought back by burnishing the spot with a smooth piece of ivory, such as is used for the handles of drawing pens. By this means, a smooth surface is obtained on which the ink will not run.

41. Erasing Shields.—In Fig. 8 is shown a brass shield that is often found useful when erasing small spots figures, or short lines. It contains holes of different shapes, so that it is possible to erase particular spots without injuring adjacent parts of the drawing. As a soiled brass shield is likely to smut the drawing, care should be taken that the shield is perfectly clean. Shields made from thick drawing paper or thin cardboard will not soil the drawing, and have the additional advantage that they may be cut to the exact size or shape of the part to be erased; on the other hand, they have the disadvantage of wearing away sooner than those made of brass.

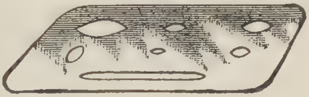


FIG. 8

42. Cleaning Drawings.—A drawing is almost sure to become soiled from the rubbing of the draftsman's sleeves, the perspiration from his hands, and from dust that accumulates on it. This may be prevented to a certain extent by covering the work, except those parts on which the draftsman is engaged, with transparent paper thoroughly secured at the edges so as not to interfere with the operation of the triangles and **T** square. After a drawing has been worked on for some time, an erasure brings out the original color of the paper, showing the amount of dirt that has accumulated.

Before commencing work, the scales, triangles, and **T** square should be thoroughly cleaned. One of the main sources of dirt on a drawing is the sliding of the instruments over the surface; celluloid and rubber triangles are especially liable to accumulate dirt. Any particles of pencil chips or rubber that may accumulate underneath the paper will raise small hills on the surface; consequently, the drawing board should be thoroughly cleaned before the paper is tacked in place. For cleaning drawings, sponge rubbers are by far the most effectual means. Though a piece of stale, well-kneaded bread will clean a drawing, or at least will distribute the dirt so that the drawing will have a cleanly appearance, it is not recommended.

43. Cleaning Tracings.—Tracings can be cleaned by means of a soft cloth soaked in benzine. This effectually removes all dirt and pencil marks without injuring either the tracing cloth or the ink lines. Care should be taken, where it is desirable to clean the tracing in this way, that the ink is a good waterproof India ink. It is always advisable to experiment with the ink on a small piece of cloth before using it on the drawing. The benzine should be pure and contain no water, as water will destroy the tracing cloth and smear the ink.

44. Lettering.—Great stress was laid in *Geometrical Drawing* on the necessity of learning how to letter neatly. The student should remember that facility in lettering is acquired only by practice, and he should not become discouraged if his lettering on the first plates is not so good as that on the Schools' plates. To further assist him in becoming acquainted with the proper method of lettering the drawings, some additional information on lettering is here given.

The lettering on the plates in *Geometrical Drawing* is frequently used on mechanical and engineering drawings. The type used for the title is known as the *block letter*, while the single-line Italic is used for all notes and explanations. The lettering of the titles on these plates, while very legible and presenting a good appearance, cannot always be executed as quickly as the exigencies of the drafting room require; therefore, some other style of lettering is usually adopted.

45. Vertical Block Letters.—The block letter illustrated and described in *Geometrical Drawing* is sometimes modified by making the corners angular instead of round, as shown in Fig. 9. All these letters can be made with the right-line drawing pen, and when carefully laid out present a very good appearance. The total width of each letter, except *A*, *G*, *I*, *L*, *M*, *W*, *Y*, and the figures *1* and *4*, is four-fifths the height. The thickness or width of *1* and *I* depends on the size of the line used, the size or thickness of line in the figure being one-fifth the height. The width of *L* is usually made a little less than four-fifths the height. The total widths of *A*, *M*, and *Y* are each made equal to the

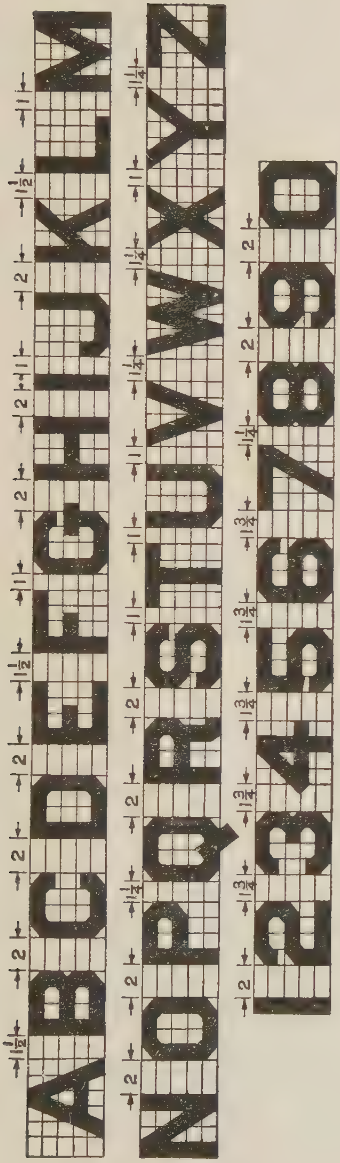


Fig. 9

height, and that of *W* is 1.1 times the height. The total widths of the letter *G* and figure *4* are each a little greater than four-fifths the height, as shown in the figure.

The widths of the spaces between the letters depend on the arrangement of the letters in the words, and are different for different arrangements. The letters in Fig. 9 were spaced to present the best possible appearance for that arrangement. The spaces in the figure are given in parts of the height, the unit being one-fifth the height. For example, the space between *B* and *C* is given as *2*, indicating that this space is 2 parts, or two-fifths of the height. It is difficult to give rules for the spacing of letters in every case that may arise; the best spacing is done by eye so as to give a smooth unbroken appearance. If the letters are too close together, they appear crowded and are difficult to read; if they are too far apart, the open spaces between them give the drawing a broken appearance. Where the letter *A* is adjacent to *F*, *P*, *T*, *V*, *W*, or *Y*, the lower extremity of *A* is placed vertically under the extremity of the other letter. The same rule applies to the letters *F*, *J*, *L*, *P*, *T*, *V*, *W*, and *Y*, whenever any of them is adjacent to another letter in such a way as to make this spacing possible. On the Practice Sheet, presently to be described, the student will lay out the letters according to the guide lines shown in Fig. 9.

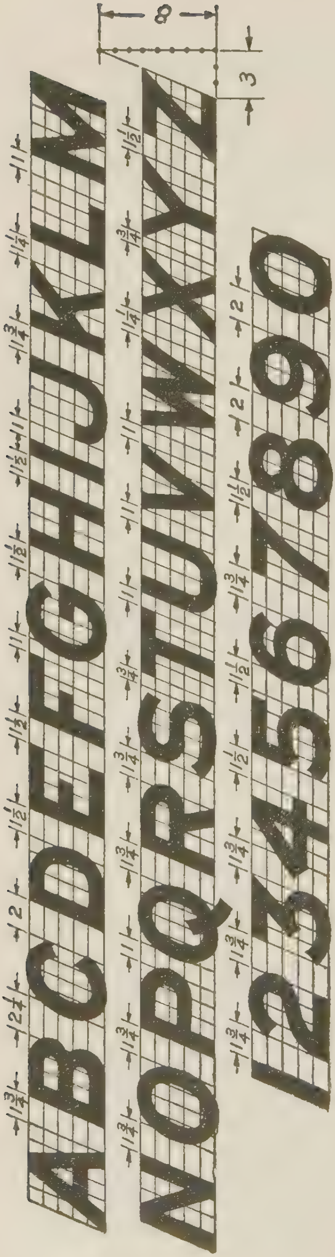


Fig. 10

46. Inclined Block Letters.—A better form of lettering for freehand work is shown in Fig. 10. The general proportions and spacing are the same as explained for the vertical letters in the preceding article. Many of the letters shown in Fig. 10 cannot be made by means of the ruling pen; hence, all should be made freehand, as a more uniform result will thus be obtained. If the letters having curved outlines are made exactly the same height as those having straight outlines, they will appear smaller; for this reason, round letters, such as *C*, *G*, *O*, are made slightly higher than the others. The inclined guide lines for these

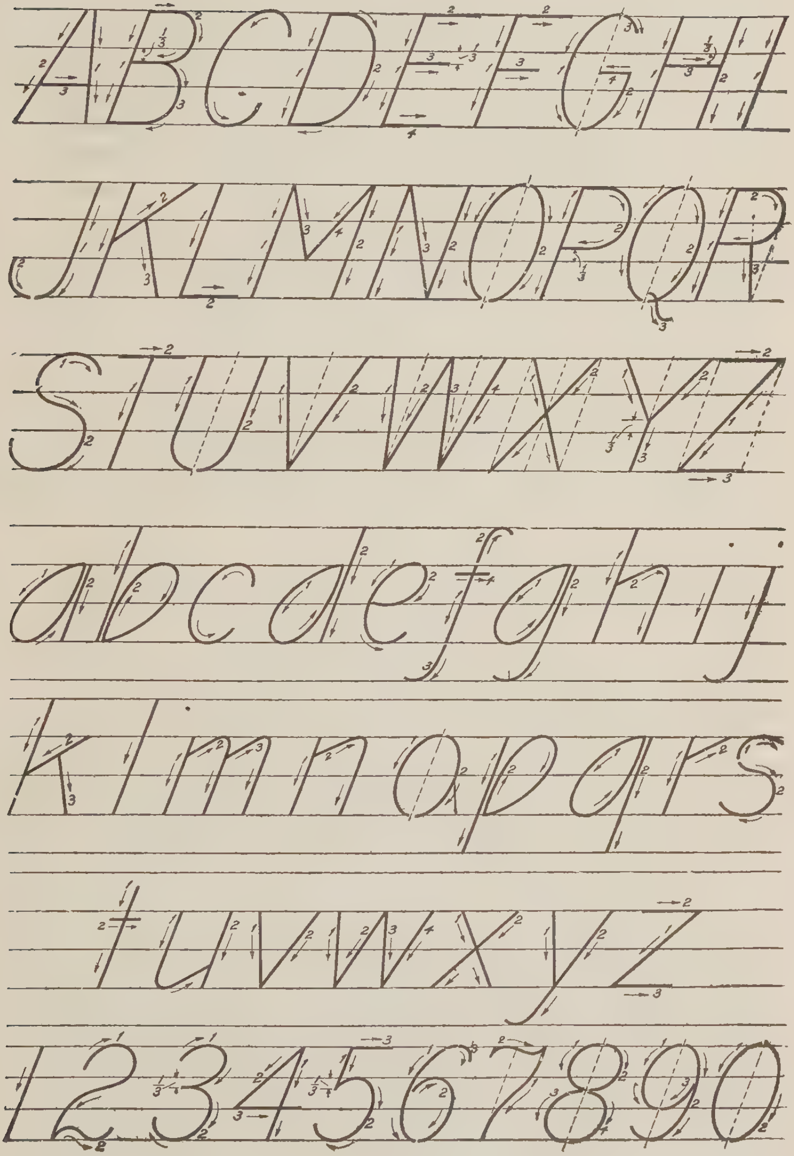

ABCDEFGHI
JKLMNOPQR
STUVWXYZ
abcdefghij
klmnopqrs
tuvwxyz
1234567890

FIG. 11

letters make an angle with the horizontal guide lines of eight vertical to three horizontal, as shown at the right end of Fig. 10, and it is a great help to rule them in, in light pencil lines, so as to keep the slope of the lettering uniform. The lines may be made almost any thickness up to nearly one-fourth the height, but clearer and more legible words are obtained if they are made not greater than one-fifth the height.

47. Working Letters.—On all drawings showing constructive features, there are numerous notes and descriptive matter. The neatness of the drawing depends to a great extent on the lettering of these notes, both as to style and execution. The style that has become general on working drawings, on account both of the rapidity with which it may be executed and of its clearness and legibility, is known as the *single stroke*, and is shown in Fig. 11. This figure shows the number of pen strokes in each letter as well as the direction and the order in which the strokes should be made. The student should observe carefully the formation of these letters and follow the order and direction of the strokes given in the figure. Each stroke is numbered, and the direction is marked by an arrow. The student is advised to practice these letters assiduously, making them at least the size shown in the figure, as, by making them large, imperfections will show more plainly and he can better see where improvement is needed.

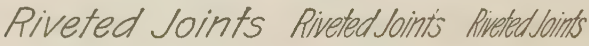
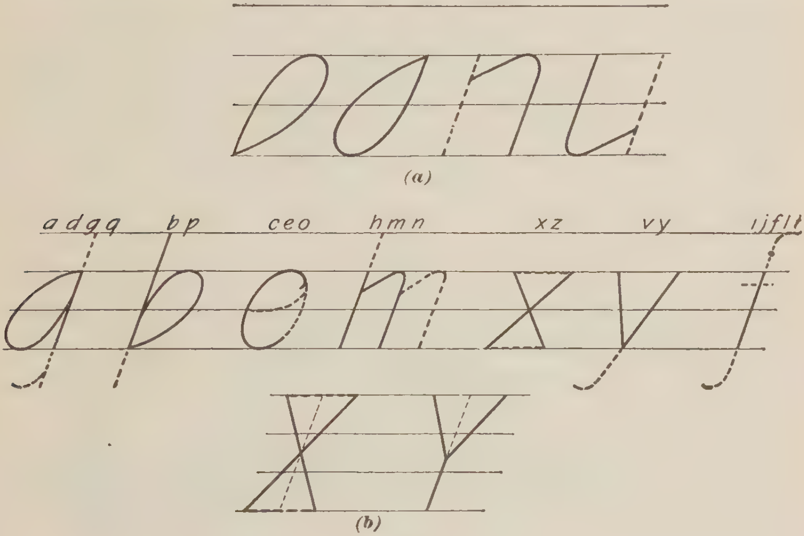


FIG. 12

By comparing this style of lettering with the single-line Italic employed for similar descriptive matter on the plates in *Geometrical Drawing*, it will be noticed how much simpler is the style shown in Fig. 11. The single-stroke letter has the further advantage of occupying less space lengthwise than any other style of letter, and, even though the letters are crowded, the legibility is not impaired, as will be observed in Fig. 12.

The width and spacing of these letters may be made in one of the ways shown in Fig. 12, according to the amount of room available.

The alphabet shown in Fig. 11 is of value to the draftsman and engineer, and the student should by continual practice familiarize himself with the formation and proportion of these letters. The elements of the letters are formed by a single pen stroke, and, when executed at a uniform angle and properly spaced, they present a line of great neatness and legibility. The principles on which these letters are constructed are shown in the ovals of Fig. 13 (*a*), as are also the characteristic curves by which such letters as *m*, *n*, and *u* are joined at the top or bottom. At (*b*), the similarity of certain letters is shown, which if carefully noted will assist in obtaining uniformity in lettering. Although a slope of 60° may be used for these letters, the



(*a*)

(*b*)

FIG. 13

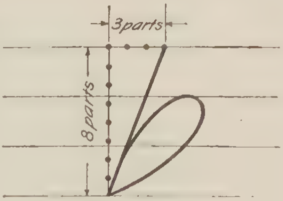


FIG. 14

best slant is that of the hypotenuse of a triangle having a base of three parts to a height of eight, as shown in Fig. 14.

In executing this lettering, the principal points to be observed are to keep the letters the same height and slant and as regularly spaced as possible. It is better, where neat lettering is required, to draw four or five guide lines for the letters, as shown in Fig. 11. The four spaces in the figure are equal; the top line limits the height of the capitals and of such letters as *b*, *d*, *f*, *h*, *k*, *l*, etc.; the second line from the top locates the height of the small letters. Such letters as *g*, *j*, *p*, *q*, and *y* extend beyond the fourth line a distance equal to one space. It is, however, unnecessary to draw a line for this purpose, as these letters are usually at such a distance from one another that a slight deviation is indiscernible and the eye is a sufficient guide. A good exercise for the student who wishes to become proficient in the execution of this type of lettering is shown in Fig. 15; it consists in drawing, freehand, numerous lines about the height of the large letters, all parallel and at the proper angle with the horizontal.

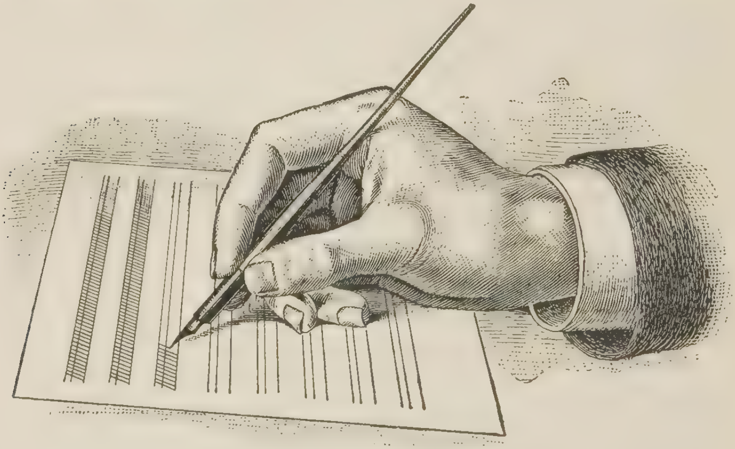


FIG. 15

There are many devices that the ingenious draftsman may adopt in order to facilitate the lettering on a drawing.

It is a common practice, where there is much descriptive matter that has not been written on the pencil drawing but must be written on the tracing, to place under the tracing cloth a sheet of ruled cross-section paper and use the rulings as guide lines for the lettering.

48. **Neatness.**—It should be constantly borne in mind that the work of a draftsman should be accurate and neat; that all lines, figures (numbers), and letters must be clear cut and distinct; that there must be no doubt as to the meaning of lines and dimensions; that mistakes made on drawings are often more serious than errors in the field or shop, for they may not be located until the various parts of the work are put together. Even though the tracer realizes, in practice, that all his work will be checked, he should not relax his vigilance on that account. If a man is inaccurate, the checker soon finds it out, and he is usually not slow in imparting the information to the chief draftsman.

49. **Sending the Work.**—The student will send to the Schools tracings of all the remaining plates of this Course. He should not send the pencil drawings, which he should retain, as it may be necessary for him to retrace them.

PLATE 101, TITLE: PRACTICE SHEET

50. **Introduction.**—This plate is intended for a practice sheet in tracing, and is introduced to give the student an opportunity to familiarize himself with the methods of making lines and letters on tracing cloth; it is not in any way a working drawing.

In order to distinguish readily between references to figure numbers on the drawing plates and those in the text, the former will be designated by a star. Thus, Fig.* 1 indicates figure 1 on the drawing plate under consideration. The usual abbreviation Fig. refers to figures in the text.

51. **Directions for Laying Out the Pencil Drawing.** Fig.* 1 is a set of lines of different kinds and weights inclined

at angles of 45° to the horizontal, and enclosed in a square 5 inches on a side. Locate the top line of the square $\frac{3}{4}$ inch from the top border line, and the left side $\frac{1}{2}$ inch from the left border line. Divide each of the four sides of the square into 19 equal parts in one of the ways described in *Geometrical Drawing*. Draw the lines from the points of division with the aid of the 45° triangle resting on the **T** square. In the pencil drawing, the lines are all drawn of the same weight, and, if desired, all full. In tracing them, the pen should be adjusted to obtain the proper widths. At first, it is better for the student not to draw full those lines that are to be composed of dots or dashes, but to draw them in pencil just as they are to appear on the tracing, with the exception of the weight or width.

52. Fig.* 2 is a series of ten concentric circles, the outer circle being 5 inches in diameter. First draw the two diameters at right angles to each other, locating the vertical diameter at the center of the plate and the horizontal diameter $3\frac{1}{4}$ inches from the top border line. The radii of the circles can be laid off along the horizontal diameter by means of the full-size scale. They are $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$, $1\frac{3}{4}$, 2, $2\frac{1}{4}$, and $2\frac{1}{2}$ inches, respectively. Do not bear on the point of the compass at the center very hard, or a bad hole will be worn in the tracing. As in the case of Fig.* 1, the pencil lines may all be made the same weight if desired.

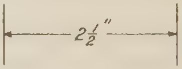
53. Fig.* 3 consists of several semicircles enclosed in a circle 5 inches in diameter. First draw the two diameters at right angles to each other, placing the vertical diameter 3 inches from the right border line and the horizontal diameter $3\frac{1}{4}$ inches from the top border line. Draw the outside circle 5 inches in diameter. Mark off the horizontal diameter into 16 equal parts of $\frac{5}{16}$ inch each by means of the full-size scale. Use each of the points of subdivision as the center of a semicircle, and draw the semicircles as shown. The radii of the semicircles are, respectively, $\frac{5}{16}$, $\frac{5}{8}$, $\frac{15}{16}$, $1\frac{1}{4}$, $1\frac{9}{16}$, $1\frac{7}{8}$, and $2\frac{3}{16}$ inches. Great care must be taken in laying off the distances along the horizontal diameter and in drawing the semicircles, that they will be exactly tangent, as shown, and meet the outside circle at the ends of the diameter. The object of this exercise is to teach the student to connect the curves by smooth lines without any breaks or irregularities.

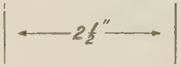
54. Fig.* 4 represents various kinds of lettering that will be used in this and succeeding plates. They are made as described in previous articles, and no further instructions are necessary, except as to their height. The letters in the heading *Letters & Figures* are $\frac{1}{4}$ inch in height; their thickness is about one-sixth the height, but the student may make the thickness equal to one-fifth the height, as in Fig. 10, if he so desires. The letters and numerals in Type I and Type II are all $\frac{1}{2}$ inch in height, and are the same as the alphabets shown in Figs. 9 and 10, respectively. The letters and numerals in Type III are $\frac{1}{8}$ inch high, and the fractions are $\frac{1}{4}$ inch high; the letters and numerals are made as shown in Fig. 11. In Type IV, the stems of such letters as *b*, *d*, and *h* are $\frac{1}{8}$ inch high, and the main parts of the letters are made two-thirds this height. In the note, the letters are $\frac{3}{32}$ inch in height, and in the last line of letters the stems are $\frac{1}{4}$ inch in height. The student is advised to draw the letters of each type on small sheets of paper several times, so as to become familiar with the method of forming them before putting them on the plate. This will give him a good idea of the amount of room to leave for each letter, so that he can arrange the different types laterally about as shown on the plate. The lower guide line of the heading is $6\frac{9}{16}$ inches above the lower border line; the lower lines of Type I are, respectively, $5\frac{3}{4}$, 5, and $4\frac{1}{4}$ inches above the lower border line; of Type II, $3\frac{1}{8}$, $2\frac{3}{8}$, and $1\frac{5}{8}$ inches; of Type III, 6, $5\frac{5}{8}$, and $5\frac{3}{16}$ inches; of Type IV, $4\frac{5}{16}$ inches; of the note, $3\frac{7}{16}$, $3\frac{3}{16}$, and $2\frac{15}{16}$ inches, and of the last line of letters, $2\frac{3}{8}$ inches. In drawing the letters on the plate, it will be best for the student to draw all necessary horizontal, inclined and vertical guide lines. He can then follow the instructions given in preceding articles in forming the letters. In practice, it is not customary to draw all the guide lines, as an experienced draftsman can usually form the letters by eye, with the aid of the lower guide line alone.

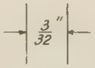
55. Fig.* 5 shows the different kinds of lines that are commonly used in mechanical and construction drawing. In drawing them on the tracing, they should in every case be made slightly heavier than shown on the plate, since the plate is somewhat reduced in size. The letters in the heading *Lines* are $\frac{1}{4}$ inch high, the same as for the heading for Fig.* 4, and they are made the same as Type II, Fig.* 4. The lower guide line of the heading is 1 inch from the lower border line. The names of the lines are $\frac{1}{8}$ inch high, the letters being the same as those of Type III, Fig,* 4. The lower guide lines for those letters are, respectively, $\frac{3}{16}$ and $\frac{1}{2}$ inch from the lower border line. Each line, including the heading, is $5\frac{1}{2}$ inches in length.

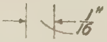
DEFINITIONS OF LINES

56. Surface lines are lines that limit or bound the delineation of an object, and should be of the same weight all through the figure.

57. Dimension lines are lines drawn to show limits to which dimensions apply, and are ended with an arrowhead, put in freehand; the arrowheads should always touch the lines that limit the dimension, thus: 

; never thus: 

. A short dimension may be indicated thus: 

, placing arrowheads outside of the limiting lines, or thus: 

Do not put dimension lines in or over the delineation of parts, nor place figures where they cannot be easily read.

58. Center Lines.—When objects are symmetrical, or nearly so, **center lines** are usually drawn at the working center of the drawing. These lines are very important, being lines of reference from which the work is to be laid out. Dimensions are frequently given from the center lines, which always serve as a guide for all dimensions, even when not used to limit the dimensions. In making working drawings, it is important that one of the center lines should be first located, as this will serve as a general line of reference. This center line is not necessarily the geometrical center of the figure; it may be simply an important line in the object drawn, from which or along which, as a basis, dimensions are laid off.

This line is also used to indicate where a section has been or is to be taken. If, however, a partial section is indicated, the remainder of the view being in elevation or plan, the full (surface) line is used, except where there is an opening appearing in both parts of the view; in this case, the dash and dot would be used where the line crosses the opening, and the full line where it crosses the solid portion.

59. **Hidden surface lines** are lines drawn to show the surfaces of hidden parts. It should be noticed that a part hidden in one view is sometimes drawn full in other views, indicating that from one position the part cannot be seen, while from the other position it can.

60. As previously stated, all lines on tracings should be fully 50 per cent. heavier than the lines used on the plates drawn in connection with *Geometrical Drawing*. The only case in which a fine line should be used is when locating points by intersecting lines, and even then they are not actually needed, since the points can be located by fine pencil lines, and being once located the lines passing through the points may be as heavy as desired. Fine lines on tracings render it very difficult to obtain a good blueprint.

61. It will be noticed that the broken-and-dotted lines, here used for center lines and to indicate where a section is to be, or has been taken, consist of a repetition of long dashes, each followed by one dot. In *Geometrical Drawing*, two dots were used. Both forms of this line are widely employed, but the student is advised to use the one here shown, as it is simpler. When working in a drafting room, however, a draftsman must conform to the standards in force there.

PLATE 102, TITLE: MASONRY AND TIMBER

62. Introduction.—This plate is intended as a practice sheet to teach the methods employed to represent different kinds of masonry, as well as masonry and timber construction.

63. Masonry Retaining Wall.—Fig.* 1 shows the front elevation, top view, and cross-section of a retaining wall of stone masonry. The principal dimensions are given, together with the names usually applied to the various parts of the wall. The vertical height of each course is given, no allowance being made for joints. This is quite common practice; the actual sizes of the stones in the different courses depend on the kind of masonry, and are usually computed by the stone cutter from the dimensions given on the drawing. In the front elevation, the longer stones (**stretchers**) are shown 3 feet long, and the narrower stones (**headers**) are shown 18 inches in width.

The three views are located on the plate as follows: The top line of the front elevation is $2\frac{1}{4}$ inches from the top border line, the left vertical cutting line is $\frac{3}{4}$ inch from the left border line, and the front elevation is $4\frac{1}{2}$ inches long. The front line of the top view is $\frac{1}{2}$ inch from the top line of the front elevation. The face of the abutment, as shown in cross-section, is **plumb**, that is, vertical, and $\frac{3}{4}$ inch from the right vertical cutting line of the front elevation.

With these lines laid out on the plate, the dimensions are then laid out to the scale of $\frac{1}{4}$ inch to the foot; that is, each foot on the wall is shown as $\frac{1}{4}$ inch on the drawing. For example, the intermediate courses or layers of stone measure each $\frac{1}{4}$ inch on the drawing, and are 1 foot thick in the actual wall. The best results will be obtained if the cross-section is laid out to scale before the remainder of the figure is drawn. The other views can then be projected as shown.

In laying out these dimensions to scale, the general instructions for scaling given in Arts. 28 and 29 must be followed. Only the outlines, joints, and dimensions need be given on the pencil drawing, the fine parallel lines shown in the interior of the cross-section being omitted on the pencil drawing and drawn on the tracing. These lines are called **section lines,** or **cross-hatching,** and are the same distance apart all over the section. There are several special appliances for ruling section lines, but in the plates of this Course all section lines will be drawn without the use of a scale or special instrument, the eye being the only guide in spacing them evenly, and the student will follow this plan in drawing the sections shown. With practice, great skill can be acquired in spacing the lines evenly, and the work can be done very rapidly.

64. Reinforced-Concrete Retaining Wall.—Fig.* 2 shows the rear elevation and cross-section of a retaining wall of reinforced concrete. The principal dimensions of the wall, together with the size and spacing of the reinforcing rods, are given in the figure. The dotted lines in the rear elevation represent the reinforcing rods; the full vertical lines are the outside surfaces of the counterforts. In the cross-section, the reinforcing rods in the counterfort are shown by dotted lines; in this view, the rods that are cut by the section are shown as small black squares, and those parallel to the section as heavy black lines; this is the usual practice for this type of construction. This view also illustrates the usual method of representing broken-stone concrete. In forming the concrete, it is customary first to draw

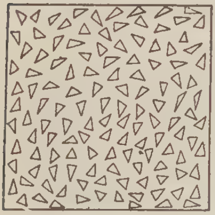
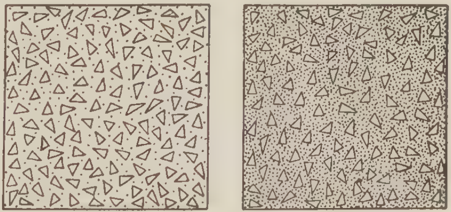


FIG. 16



(a) FIG. 17 (b)

at random throughout the section a number of irregular triangles, as shown in Fig. 16; these are assumed to represent the irregular broken stones. When a sufficient number have been drawn, the remaining area, between the stones, is filled with small dots. The section can be shown light by putting in only a few dots, as in Fig. 17 (*a*), and dark by showing a large number of dots, as in Fig. 17 (*b*). The student should try to get approximately the same shade as that used in the plate. For this kind of work, a Gillott's 303 or some other kind of fine pen should be used.

65. When gravel is used in the concrete instead of broken stone, it is customary first to draw a number of irregular rings or closed figures, as shown in Fig. 18. The remaining area is then filled in with small dots the same as for broken-stone concrete (see Fig. 19).

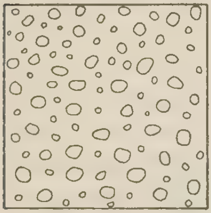
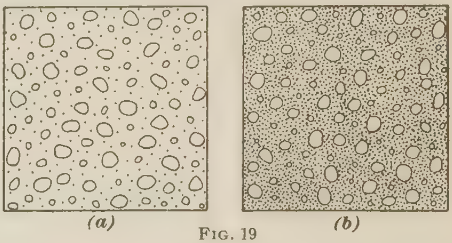


FIG. 18



(*a*) (*b*)

FIG. 19

66. The two views in Fig.* 2 are located on the plate as follows: The bottom line of the rear elevation is 1½ inches above the lower border line; the left vertical cutting line is ¾ inch from the left border line, and the rear elevation is 4 inches wide. The face of the retaining wall, shown in the cross-section, is plumb, and is located 7¼ inches from the left border line. With these lines located on the plate, the remaining lines are laid out, according to the dimensions given, to a scale of ¼ inch to the foot. It is the best plan to lay out the entire cross-section before drawing the rear elevation; then the lines can be projected across from the section to the elevation.

67. Brick Masonry.—Fig.* 3 shows the front elevations of portions of brick walls laid in different styles.

The first portion on the left is called **English bond.** In this style, the courses are alternately stretchers (that is, bricks with the long side at the surface of the wall) and headers (that is, bricks with the short side at the surface of the wall).

The second portion is called **Flemish bond.** In this style, the bricks in every course are alternately headers and stretchers.

The third portion is called **common bond.** In this style, five to ten courses are laid with stretchers, and then one course with headers.

The standard form of brick measures about $8\frac{1}{4}$ in. × 4 in. × $2\frac{1}{4}$ in., and the joints are, as a rule, $\frac{1}{4}$ inch thick. The lines representing the joints allow for the thicknesses of the joints, and are, respectively, $2\frac{1}{2}$, $4\frac{1}{4}$, and $8\frac{1}{2}$ inches apart. They are laid out to a scale of $\frac{1}{2}$ inch to the foot, the top line of each being 1 inch from the top border line. The student can arrange the three parts of the figure on this line about as shown in the plate.

68. Squared-Stone and Ashlar Masonry.—Fig.* 4 shows the front elevation of three kinds of squared-stone masonry. In **range masonry**, all the horizontal joints are continuous; in **broken range**, the horizontal joints are partly continuous; in **random range**, there is no continuity in the horizontal joints. The top lines of these figures are $3\frac{1}{2}$ inches from the top border line; the figures are $1\frac{5}{8}$ inches high. The student can arrange the joints and also the figures on the sheet approximately as shown on the plate. There is no need, in the case of the broken-range and random-range masonry, for the student to arrange the joints exactly as shown on the plate. It will be better practice for him to exercise his ingenuity and arrange them to suit himself.

69. Rubble Masonry.—Fig.* 5 shows two kinds of rubble masonry. In the **uncoursed** rubble masonry, no joints, either vertical or horizontal, are continuous; in the **coursed** rubble masonry, the stone is finished off horizontal at occasional intervals. The top line of each figure is

6⅛ inches from the top border line, and each figure is 1⅜ inches high. The student should follow the same general arrangement as shown on the plate when he arranges the joints, but there is no need to copy or reproduce them exactly. To get the best results, these joints should be drawn in freehand.

70. Timber Retaining Wall.—Fig.* 6 shows the front elevation and cross-section of a timber retaining wall, and illustrates the usual methods of representing the end and side views of timber. The top line of this wall is 4½ inches, and the bottom line 2 inches above the lower border line. The front line of the cross-section is plumb, and is 3 inches from the right border line. This style of wall is frequently used for temporary purposes, and when so used is most often composed of second-hand railroad ties or other second-hand material. It should not be considered in connection with permanent work. The timbers are approximately 6 in. × 8 in. × 8 ft., framed to 4 inches thick where the transverse pieces enter the face of the wall. The dimensions given on the plate are laid off to a scale of ½ inch to the foot.

71. Concluding Remarks.—It is a waste of time to put cross-section lines, conventional signs for concrete, the wood marks on timber, etc. on pencil drawings, as better results can usually be obtained if these are made directly on the tracings. All dimension lines, dimensions, notes, etc. should be put on the pencil drawing to assist in locating them properly when tracing. It is also well in the beginning to draw guide lines for all dimensions and letters. The heights of letters on this and succeeding sheets were chosen so as to give the letters the proper appearance with relation to the remainder of the drawing. The student should try to make his letters about the same height as those shown on the plate, but there is no necessity for him to make them exactly the same height.

PLATE 103, TITLE: STRUCTURAL STANDARDS

72. Introduction.—This plate shows some structural standards much used in civil-engineering work. Figs.* 1 to 10 are cross-sections of standard structural shapes; Figs.* 11 to 15 show portions of other standards.

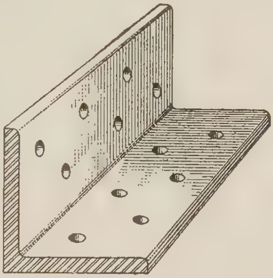


Fig. 20

73. Angles.—The shapes shown in cross-section in Figs.* 1 and 2 are called **angles,** and are usually designated by the capital letter **L**. The two sides are at right angles and are called the **legs.** Fig. 20 shows a perspective view of an angle, together with the holes that are used for riveting the angle to another angle or some other piece of steel. The portions in Figs.* 1 and 2 with no cross-hatching represent open holes for rivets. The lower line of each angle is $2\frac{1}{4}$ inches below the top border line. The left line of Fig.* 1 is 1 inch, and that of Fig.* 2 is $3\frac{7}{8}$ inches from the left border line. The dimensions given on the plate are drawn to a scale of 3 inches to the foot.

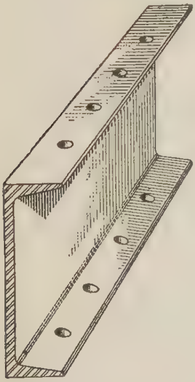


Fig. 21

74. Channels.—The shapes drawn in Figs.* 3, 4, and 5 are called **channels,** and are usually designated by the symbol **[**. Fig. 21 shows a perspective view of a channel, together with the holes that are left for rivets. The narrow vertical portion is the **web;** the remainder (the horizontal portions), are the **flanges.** The outer surfaces of the flanges are at right angles to the web. The inner surfaces of the flanges are not parallel to the outer surfaces, but slope, as shown on the plate, with a slope of 6 to 1. The top line of each channel is $\frac{5}{8}$ inch below the top border line; the left line of Fig.* 3 is $6\frac{3}{8}$ inches, that of Fig.* 4 is $8\frac{3}{8}$ inches, and that of Fig.* 5 is 10¼ inches from the left border line. The lines given on the plate are drawn to a scale of 3 inches to the foot.

75. Z Bars.—The shapes shown in Figs.* 6 and 7 are called **zee bars**, and are usually designated by the symbol **Z**.

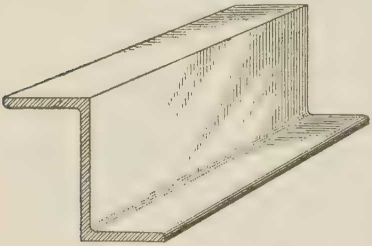


FIG. 22

Fig. 22 shows a perspective view of a **Z** bar. The vertical portion is the **web**; and the horizontal portions are the **flanges**. The flanges are at right angles to the web. In both Figs.* 6 and 7, the top line is 4 inches from the top border line; the left side of the web of Fig.* 6 is 1¾ inches, and that of Fig.* 7 is 4½ inches from the left border line. The lines given are drawn to a scale of 3 inches to the foot.

76. I Beams.—The shapes shown in Figs.* 8, 9, and 10 are called **I beams**, from the similarity of the cross-section to the capital letter I. The vertical part is the **web**; the horizontal parts are the **flanges**. Fig. 23 shows a perspective view of an **I** beam, together with the holes in the top and bottom flanges. The outer surfaces of the flanges are at right angles to the web; the inner surface of the flanges are not parallel to the outer surfaces, but are inclined at a slope of 6 to 1, as shown on the plate. The lower line of each **I** beam is 1¼ inches from the lower border line; the center of the web in Fig.* 8 is 1¾ inches; in Fig.* 9, 4¾ inches, and in Fig.* 10, 8 inches, from the left border line. The shapes are drawn to a scale of 3 inches to the foot.

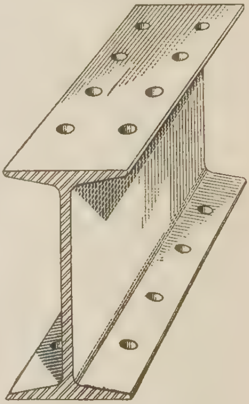


FIG. 23

77. Rivets.—Fig.* 11 shows the elevation of a steel rivet. The round head at the left end is called a **button**

head. The flat conical head at the right end is called a **countersunk head.** Most rivets have button heads at both ends, some at only one end, while a few have countersunk heads at both ends. The rivet given on the plate is drawn full size; the center of the curved outline of the bottom head is found by trial, so that the head will be $\frac{9}{16}$ inch in height and $1\frac{1}{4}$ inches in diameter. The center line of the rivet is $1\frac{1}{8}$ inches from the top border line, and the right end is $\frac{1}{2}$ inch from the right border line.

78. Eyebar Head.—Fig.* 12 shows an eyebar head. The bar is rectangular in cross-section through the greater part of its length, and is increased in width at the end so as to form a circular head. A hole is bored in the center of this head to allow the connection of the eyebar to other pieces. The center line of the bar is $4\frac{3}{8}$ inches from the top border line; the center of the circular head is $2\frac{1}{4}$ inches from the right border line. The left end of the eyebar is shown broken off by an irregular line about $6\frac{1}{4}$ inches from the right border line. The right side of the cross-section is 7 inches from the right border line. The figure is drawn to a scale of 3 inches to the foot.

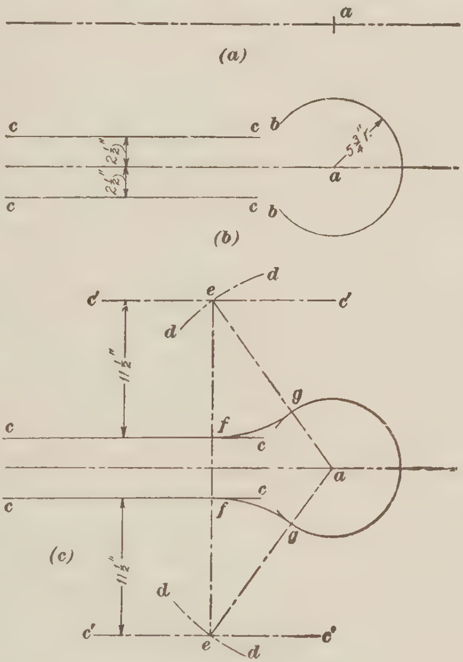


FIG. 24

The centers of the circular curves that connect the straight sides of the bar with the outline of the circular head can be found as follows: First draw the center line of the bar, Fig. 24 (*a*), and mark on it the location of the center a of the circular head. With a radius of $11\frac{1}{2} \div 2 = 5\frac{3}{4}$ inches, to a scale of 3 inches to the foot, and a as a center, draw the portion $b\,b$ of the outline of the circular head, as shown in Fig. 24 (*b*). Lay off the width of the bar, one-half on each side of the center line, and draw the lines $c\,c$ parallel to the center line representing the edges of the bar. Lay off, above and below the lines $c\,c$, distances of $11\frac{1}{2}$ inches, and draw the construction lines $c'\,c'$, Fig. 24 (*c*), parallel to $c\,c$. With a as a center, and a radius equal to $11\frac{1}{2} + 5\frac{3}{4} = 17\frac{1}{4}$ inches, describe the circular arcs $d\,d$, intersecting the lines $c'\,c'$ at e. Then the points marked e are the centers of the curves. Draw the line $e\,e$; this line intersects $c\,c$ at f, the beginning of the curve. Draw the lines $e\,a$; these lines intersect the circle $b\,b$ at g, g, the points where the curves join. With the points e as centers and a radius of $11\frac{1}{2}$ inches, the arcs $f\,g$ can now be drawn.

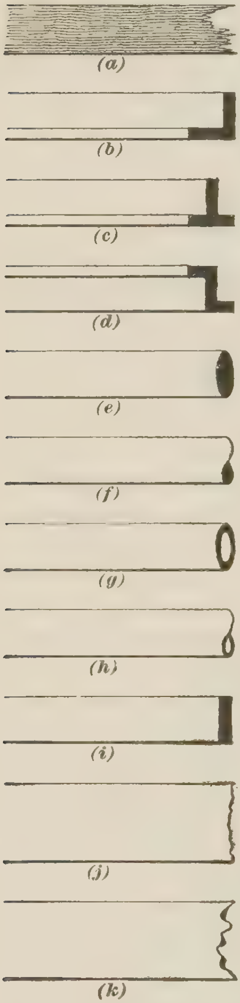


FIG. 25

79. Breaks.—In Fig.* 12, the eyebar is shown broken off at the left end. This is frequently done when it is not desired to show the whole of a part. In many cases, the end is so broken or finished as to indicate the approximate shape of the object broken. Conventional methods of indicating breaks are shown in Fig. 25. Wood is usually shown broken in the manner illustrated at (*a*), angles as at (*b*), **T**'s as at (*c*), **Z** bars as at (*d*). Cylindrical objects are occasionally broken as shown at (*e*), but most frequently in the manner shown at (*f*). Pipes and similar hollow cylindrical objects may be broken as shown at (*g*); but more frequently the break is made as shown at (*h*). Rectangular objects may be broken in the manner shown at (*i*). Plates and objects other than those included between views (*a*) and (*i*) are often shown broken off by drawing a wavy freehand line, as in (*j*) and (*k*). These methods will be followed in the subsequent plates of this Course.

80. Lateral Pin With Cotter.—Fig.* 13 is a side view of a **lateral pin** with a **cotter** in the end. This pin is circular in cross-section and somewhat resembles a bolt with a large diameter. The diameter at the right end is about $\frac{1}{4}$ inch less than the body of the pin—that is, about $2\frac{3}{4}$ inches. The center line of the pin is $6\frac{3}{4}$ inches below the top border line; the right end is $5\frac{3}{8}$ inches from the right border line. The figure is drawn to a scale of 3 inches to the foot.

81. Pin Nut.—Fig.* 14 is a top view and side view of a hexagonal pin nut. The center of the top view is $7\frac{1}{8}$ inches from the top border line, and $2\frac{3}{8}$ inches from the right border line. The right side of the side view is $\frac{5}{8}$ inch from the right border line. The figure is drawn to a scale of 3 inches to the foot. In drawing the hexagonal outline, the directions given in *Geometrical Drawing* for laying out a hexagon should be followed.

82. Loop-Welded Rod.—Fig.* 15 shows a cross-section and views of two sides of the end of a square **loop-welded rod.** The loop end is formed by bending the end of a straight rod into a circle and welding the end to the main part of the rod. No difficulty will be experienced in laying this out on the drawing. A scale of 3 inches to the foot should be used. The center line of the upper view is $3\frac{3}{8}$ inches above the bottom border line; the center of the lower view is $2\frac{1}{4}$ inches above the border line; the center of the cross-section is $6\frac{5}{8}$ inches, and the center of the circular portion of the end is $1\frac{3}{4}$ inches, from the right border line.

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